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AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

THE DESIGN OF ROCK-FILL DAMS

By J. D. Galloway,[1] M. Am. Soc. C. E.

Synopsis

The design of rock-fill dams is discussed in this paper. The term is defined, and a short history is given of the origin and evolution of the type, together with a description of some of the important dams that have been built. The major elements of a rock-fill dam include the foundation, the nature of the rock of which the dam is constructed, the dimensions of the loose rock-fill, settlement, the impervious element on the water face, the intermediate rubble cushion, and expansion joints. The limiting conditions under which such dams should be built are considered.

During the past 75 years there has been developed, in California and other Western States, a dam of a distinctive type which is of sufficient importance to warrant description. This is the rock-fill dam, which in recent years has been built to heights such as 270 ft at Dix, in Kentucky, or 328 ft at Salt Springs, in California. Although there is a general agreement among engineers as to the principal features of this type of dam there are sharp differences of opinion regarding details of construction. It has been thought desirable to give a description of the elements of rock-fill dams in order that the present stage of this evolution may be presented and the differences among engineers discussed.

The description includes a definition of the term, "Rock-Fill Dams," an historical sketch of the evolution of the type, with details of construction, of some representative dams, the elements of the design, the material and methods used in construction, the limiting conditions under which such dams may be built, and the behavior of the dam after completion and when subjected to the water load.

Definition and Principles

The principles governing the design of rock-fill dams are the result of an evolution from the first ones built by the hydraulic miners in California, following the discovery of gold in 1848. The general form in plan and cross-section,

Note.—Discussion on this paper will be closed in February, 1938, *Proceedings*.

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as far as design is concerned, is almost entirely empirical. Various engineers have introduced different elements into the design until a fairly uniform cross-section has been evolved. Some dams have failed, thus furnishing information as to what should not be done. Many have been constructed that have given years of service and are to be considered entirely safe. It must be borne in mind, however, that although the design of rock-fill dams is mainly based upon experience certain definite theoretical principles apply to such structures which should always be considered.

Rock-fill dams do not represent the most stable and certain type, but there are a number of reasons for their use. They are generally built in relatively remote locations where the engineer uses the material at hand for his structure. In such locations the cost is usually less than for other types, such as gravity or buttressed dams. Foundation conditions are often a determining element in selecting a rock-fill dam as the best type to be built. Judgment, which determines so much in engineering, must be depended upon in the final decision as to what type to adopt.

Rock-fill dams were evolved from the log crib dam in which a series of log cribs, filled with rock and faced with timber, formed the dam. Such structures have been built in many countries where timber is, or was, abundant. Many dams of the log crib type were built throughout the United States in the period of early development, the height ranging from 5 ft to 25 ft.

Definition.—Rock-fill dams, as now built, are composed of three elements: A loose rock-fill forming the mass of the dam; an impervious face next to the water; and a rubble cushion between the two.

The characteristic that differentiates rock-fill dams from other types is that the element resisting the thrust of the water pressure is of loose rock of varying sizes, placed as a fill at an appropriate site. In almost every case, the rock is dumped loosely in position and there is no attempt at orderly arrangement of the individual rocks; nor is there any other material introduced to bind the rocks together. The mass of rock is somewhat consolidated when placed in position and further consolidation takes place by settlement under load and the action of the weather.

Resistance to the forces imposed by water is obtained from the weight of the mass of rock in the dam. There can be no arch action; nor can there be any action such as the cantilever effect of a gravity masonry dam.

As the mass of loose rock permits the free passage of water it is necessary to provide the dam with an impervious element to make it water-tight. Several different arrangements have been tried. A combination of an earth fill backed by a rock-fill has been used, but this arrangement is not properly a rock-fill dam. In a few cases a diaphragm has been placed in the center of the dam, examples being the old Lower Otay Dam in California, with its steel diaphragm encased in concrete, or the Crane Valley Dam (California) with its masonry core wall. The most usual arrangement is to place the impervious element as a facing on the water side of the dam. This facing in the earlier dams was made of timber, but in the larger and more recent structures it has been made of concrete, usually reinforced. Sheet steel has also been used. On one structure, the Morena Dam, in California, the facing was made of uncoursed rubble

masonry set in concrete. Asphalt concrete has also been proposed for this purpose.

Early in the history of rock-fill dams it was recognized that on account of the unequal settlement of the rock mass, it was necessary to interpose between the facing and the rock-fill a layer of dry uncoursed rubble laid up by hand and derrick. This rubble acts as a semi-rigid element as it is subject to less settlement than the rock-fill and permits a better distribution of the water load to the rock-fill. The dry rubble is now recognized as one of the three important elements in the rock-fill dam.

Historical

As the design of rock-fill dams, as now practiced, has been the result of an evolution, it is of interest to sketch briefly the history of the principal dams in order to illustrate the empirical methods used. Table 1 gives a list of some of the important rock-fill dams.

First Bowman Dam, California.—The first dams built by miners in California were log crib dams of the type with which they were familiar in their Eastern homes. Probably the highest one was the English Dam on the Middle Yuba River, more than 100 ft high. It was built in 1856 to a height of 79 ft,

Fig. 1.—First Bowman Dam; Constructed in 1876 and Removed in 1926

enlarged and repaired in 1876, and destroyed in 1883. It preceded the Old Bowman Dam on Canyon Creek, a tributary of the South Yuba River, built by the late Hamilton Smith, Jr., M. Am. Soc. C. E. The first part of the Old Bowman Dam was constructed in 1872 as a log crib dam about 70 ft high. In

TABLE 1.—IMPORTANT ROCK-FILL DAMS IN THE UNITED STATES

Item	Dam	River	State	Date constructed	Height, in feet	Crest length, in feet	Ratio of Horizontal to Vertical		Facing	Thickness of Rubble, in Feet		Volume, in cubic yards
							Down stream	Water face		At bottom	At top	
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
1	First Bowman	South Yuba	California	1872–76	100	425	1 : 1	1 : 1	Timber	18	6	55 000
2	First Fordyce	South Yuba	California	1873	75	650	1 : 1	1 : 1	Timber	..	..	....
3	French Lake	South Yuba	California	1873	68	250	0.5 : 1	....	Timber	..	..	....
4	Walnut Grove	Hassayampa	Arizona	1887–88	110	400	1 : 1.55	1 : 2.35	Timber	14	4	46 000
5	Castlewood	Cherry Creek	Colorado	1890	70	600	1 : 1	1 : 10	Rubble	6	4	....
6	Chatsworth	Mormon Creek	California	1896	41	159	1 : 1.15	1 : 1.75	Concrete	2		6 025
7	Escondido	San Elijo Creek	California	1895	76	380	1 : 1 and 1.25 : 1	1 : 2	Timber	15	5	37 159
8	Lower Otay	Otay	California	1894–97	130	615	1.5 : 1	1.5 : 1	Steel core	None		....
9	East Canyon	East Canyon Creek	Utah	1899–1902	93	160	2 : 1	¾ : 1	Steel core	None		39 000
10	Bear River	Mokelumne	California	1900	80	748	1 : 1; 0.75 : 1; and 0.5 : 1	0.75 : 1 and 0.5 : 1	Timber	16	8	43 543
11	Meadow Lake	Mokelumne	California	1899–1903	61	775	0.5 : 1 and 1 : 1	0.75 : 1 and 0.5 : 1	Timber	6	7	46 148
12	Crane Valley	San Joaquin	California	1910	130	1 880	1.3 : 1	2 : 1	Concrete core	..	..	....
13	Sabrina	Bishop Creek	California	1907–09	70	1 065	1.25 : 1	0.75 : 1	Timber	..	..	47 023
14	South Lake	Bishop Creek	California	1909–10	80	650	1.25 : 1	0.75 : 1	Timber	..	..	74 759
15	Morena	Cottonwood	California	1909–12	167	520	1.5 : 1	9 : 10 and 1.2 : 1	Rubble in cement mortar	50	16	306 000
16	Relief	Stanislaus	California	1907–10	140	505	1.5 : 1	0.5 : 1	Reinforced concrete	108	13	136 994
17	Cucharas	Cucharas	Colorado	1911	125	550	1.5 : 1	0.5 : 1	Reinforced concrete	30		195 000
18	Swift	Birch Creek	Montana	1914	165	..	1.25 : 1 and 1.5 : 1	1 : 1	Reinforced concrete	6	4	....
19	Strawberry	Stanislaus	California	1913–16	140	612	1.5 : 1	1.2 : 1 and1 : 1	Reinforced concrete	16	4	325 640
20	Beaver Park	Beaver Creek	Colorado	1914	87	370	1.5 : 1	0.5 : 1	Reinforced concrete	5		....
21	Dix	Dix	Kentucky	1924–25	270	1 032	1.4 : 1	0.5 : 1	Reinforced concrete	22	7	1 885 000
22	Second Fordyce	Yuba	California	1925–26	140	410	1.3 : 1	1 : 1	Reinforced concrete	6	4	417 000
23	New Bowman	Yuba	California	1926–27	168	700	1.5 : 1	0.75 : 1 and 0.5 : 1	Reinforced concrete	20	5	300 000
24	Bucks	Feather	California	1926–28	118	1 200	1.5 : 1	1.4 : 1	Reinforced concrete	7	3	347 000
25	Salt Springs	Mokelumne	California	1928–30	328	1 300	1.4 : 1	1.3 : 1	Reinforced concrete	15		3 171 500
26	Bonito	Rio Bonito	New Mexico	1931	102	440	1.4 : 1	7 : 6	Reinforced concrete	20	11	140 000
27	San Gabriel No. 2	San Gabriel	California	1932–33	280	600	1.5 : 1	1.35 : 1	Laminated concrete	15	6	1 200 000
28	Skaguay	....	Colorado	1901	70	405	0.8 : 1	0.5 : 1	Steel	..	..	....
29	Penrose-Rosemount	Beaver Creek	Colorado	1932	100	580	1.4 : 1	0.5 : 1	Steel	12.5	4	....

1876, the dam was raised to a height of 100 ft. The log crib portion was left in place, but the crib idea was definitely abandoned in favor of a loose rock-fill with a dry rubble wall on both faces (see Fig. 1). The timber facing was extended to the top of the dam, being held by log stringers fastened to the rubble wall.

This dam in its major part represented the modern plan of rock-fill dams. The dry rubble wall on the down-stream face, which retained the interior fill, may have been built as a measure of economy owing to the cost of blasting the rock. The published plans of this dam show the down-stream rubble wall of a thickness varying from 14 ft to 5 ft, but when the dam was removed in 1926, the rubble was found to be only 2 ft to 3 ft thick. The loose rock was small in size and earth and quarry waste was mixed with it. The timber logs of the old crib were found in good condition and the wrought-iron drift-bolts showed little rust. The dam stood for more than 50 yr, being replaced in 1926 by a higher dam.

French Lake and First Fordyce Dams, in California.—At about the same time the Bowman Dam was built, two other rock-fill dams were constructed on the South Yuba River in California. The Eureka or French Lake Dam was built by dry rubble stone with a slope of 1 : 1 on the water face and about 0.5 : 1 on the down-stream face. There may be some loose rock in the heart of the dam. The water face was covered with timber. This dam is still in service (1937) after a life of more than 60 yr.

The First Fordyce Dam, about 75 ft high, was built of dry rubble, about 60 ft thick at the base and with slopes of about 0.25 : 1. The facing was of timber. This dam was in service about 40 yr. In 1911 an earth and loose rock-fill was placed in front of the old dam, a concrete cut-off wall built in the fill, and a concrete face on a slope of 1 : 1 substituted for the timber face. In 1926, the dam was enlarged to a height of 140 ft by a loose rock-fill on the down-stream side with a slope of 1.3 : 1, the concrete water face being continued on the slope of 1 : 1.

The aforementioned three dams (Items (1), (2) and (3), Table 1) represent a change from the older log crib type to one in which the rock-fill alone resisted the forces of the water.

Walnut Grove Dam, Arizona.—This dam on the Hassayampa River, in Arizona, was built in 1888 of dry rubble with a narrow base and steep slopes. It was 110 ft high. The section was too frail, but apparently it held water for two years. It was washed out in February, 1890, in a flood in which the dam was overtopped by the water, the spillway being too small.

Escondido Dam, California.—This dam on San Elijo Creek, in California, built in 1895 as part of an irrigation system, was probably the first of the best rock-fill type. A loose rock-fill supported an ample dry rubble wall which was faced with timber. The slope on the water face was made 0.5 : 1 and on the down-stream face, 1.25 : 1 and 1 : 1. The dam is still in service (1937).

Lower Otay Dam, California.—This dam, 130 ft high, on Otay River, in California, was built at the same time as the Escondido Dam. The dam was designed with ample slope,[2] but the impervious element was a membrane of

[2] *Engineering News*, March 10, 1898.

steel plates riveted together, enclosed in concrete, and connected to a cut-off wall on the bottom and sides of the canyon. The membrane was placed directly in the center of the dam. The dam was in service for about 18 yr when it was overtopped in a flood on January 27, 1916, and washed out. The spillway was insufficient to carry the flood.

Bear River and Meadow Lake Dams, California.—These two dams on the Mokelumne River, in California, built 1900–1903, were about the last rock-fill dams to have the down-stream face protected by a dry rubble wall. There is a saving in the quantity of loose rock by the method, but on account of higher unit costs of the paving of the down-stream slope, it makes a more expensive dam. Fig. 2, a view of the Bear River Dam, illustrates a section common to early dams, in which both faces were laid up in rubble. The section of such dams was too frail. In one flood this dam was overtopped about 18 in. and some damage was done. Later, it was reinforced with a loose rock-fill on the down-stream side.

Some preliminary plans made by the writer for the Sabrina and Hillside Dams, on Bishop Creek, California, included a rubble facing for the down-stream slope which was not used when the dams were built. The original timber facing of Sabrina Dam is shown in Fig. 3, revealing the distortion due to settlement at places where the facing was laid on loose rock-fill. In 1929, after 20 yr in service, the facing was replaced by laying timber sleepers, between rubble, on the rock-fill.

Relief Dam, California.—This dam on the Stanislaus River, California, 140 ft high, marked what is believed to be the first use of a reinforced concrete facing. In 1906, when it was planned, Charles D. Marx, Past President and Hon. M. Am. Soc. C. E., and the writer were of the opinion that a steel-plate membrane should be enclosed in the concrete. The experiment of depending upon concrete, 36 in. thick, under a 140-ft head of water, was tried and was entirely successful.

The dam has a dry rubble wall, 100 ft thick at the bottom and 13 ft at the top, the front 5 ft being very carefully laid up and 2 ft thick next this concrete set in cement mortar. In the light of subsequent experience, these dimensions of the dry rubble seem excessive.

Morena Dam, California.—The Morena Dam, built in 1909–1912, has a height of 167 ft above the stream bed, but the cut-off wall was carried 112 ft below in a narrow gorge in the granite. The dam is of ample proportions and is noteworthy as having a facing made up of rubble stone laid in concrete 6 ft thick at the stream-bed level and decreasing to about 3 ft at the top. In addition, a layer of reinforced concrete 1 ft thick was placed on the rubble masonry rising to a point 42 ft above stream bed, above which the rubble masonry is relied upon for water-tightness.

The dam was raised 5 ft in 1916–17 and 10 ft in 1922–1923. At one time it was overtopped, but it withstood the water without damage.

Recent Dams.—Table 1 gives a list of dams with main dimensions, which have been built in the past twenty-five years. Practically all these dams are of the same type as defined above, the result of the evolution of the previous

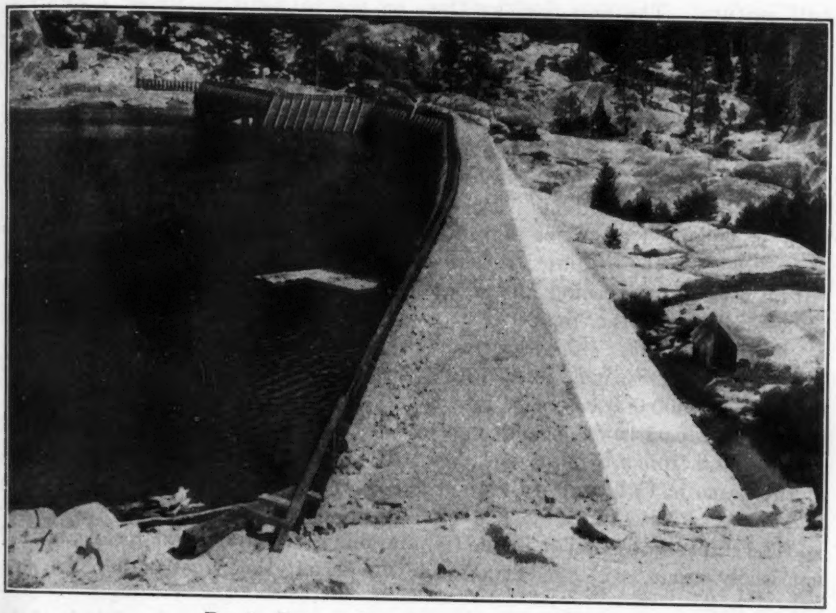

FIG. 2.—VIEW OF BEAR RIVER DAM, CALIFORNIA

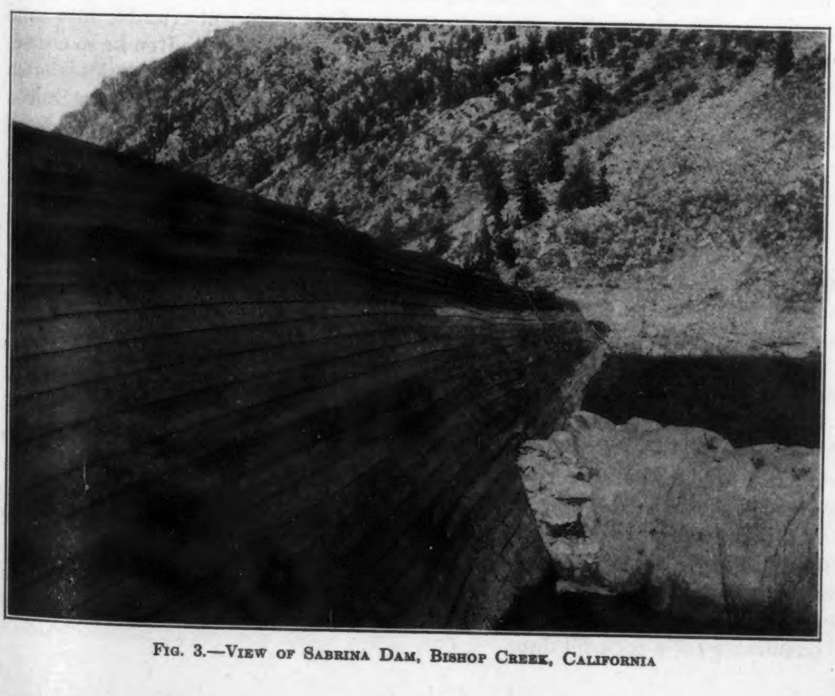

FIG. 3.—VIEW OF SABRINA DAM, BISHOP CREEK, CALIFORNIA

half century. The Salt Springs Dam on the Mokelumne River, California, 328 ft in height, represents the present maximum height of rock-fill dams.

Design

The design of rock-fill dams is based upon the empirical knowledge gained in the past. The important elements are as follows.

Foundation.—To a great extent, the criteria that obtain for other dams may be applied to rock-fill dams. Rock is the best foundation but hard material, such as cemented gravel or indurated sand, will sustain the weight of such a structure. On the assumption that water is kept from the foundations by the cut-off wall and the facing, the nature of the foundation rock is not of as great importance as in other dams.

Usually, the dam site is covered with stream gravel and boulders and with hill wash. Salt Springs Dam rests upon granite, but it was necessary to remove about 400 000 cu yd of gravel and boulder over-burden. Bucks Dam rests in one part upon granite and at the north end upon a hard indurated granite sand from which an overburden 10 to 20 ft thick was removed. Most of the dams in California rest upon native granite, as they were built in the higher Sierra Nevada.

The important element in the foundations is to obtain a material that will not easily erode, will not settle, and will permit the construction of a safe cut-off wall.

Rock-Fill.—The character of the rock from which the fill is made is of the utmost importance. In granite regions the rock varies from a hard durable stone capable of standing great stresses to stone that will crumble into sand either under pressure or the action of weather. Granite will often be so coarse-grained that freezing water will disintegrate it. Hard durable granite is often found close to belts or zones where the crystalline structure has been broken down by stresses resulting from earth movements. It is not an unusual circumstance to find a hard rock cliff of excellent rock, behind which is a mass of rock wholly unsuited for the purpose. In large dams the opinion of a competent geologist familiar with rocks should be obtained from time to time.

Other igneous rocks are sometimes suitable for rock-fill dams and often they are not. Andesites that have been completely metamorphosed into hard tough greenstones, or the diorites, are good material, but the andesites may be of such recent origin as to be unsuitable.

Sedimentary rock, such as sandstone or limestone, may furnish material suitable for a rock-fill dam. The Dix Dam was made of limestone. The writer is unaware of any dam built of sandstone.

The criterion of rock for the fill is that it shall be sound and not subject to rapid disintegration under weather conditions. Many rocks, while apparently sound, are defective in that they are cut by numerous shear planes due to block faulting or other earth stresses. The rock in a dam of even moderate size must resist considerable stresses due to the weight of the rock above and, in addition, the forces arising from the weight of water. Under such conditions any rock, however hard, that will split, crush, or otherwise disintegrate, is unsuitable for a rock-fill dam.

The nature of the rock-fill is one upon which difference of opinion will develop. It is believed that the fill should be composed of individual rocks of fairly uniform size, one rock bearing directly upon another, usually expressed as "rock to rock." Any wide divergence in size will cause excessive and unequal settlements, something to be avoided wherever possible. To illustrate the idea, if one selects a sample from a mass of broken stone of fairly uniform size, it will be found extremely difficult to force the sample into the mass by added pressure. On the contrary, if a large rock, which may rest upon small gravel or sand, is given a load, it will settle into the mass by displacing the smaller grains. In the usual method of dumping rock into the fill the larger ones roll to the bottom and the smaller ones remain at the top. A graded fill results. At Salt Springs, rocks as heavy as 25 tons were used, but the usual size was about 10 tons. At Bucks (see Fig. 4), where the rock was handled entirely by trucks, the maximum size was about 3 tons.

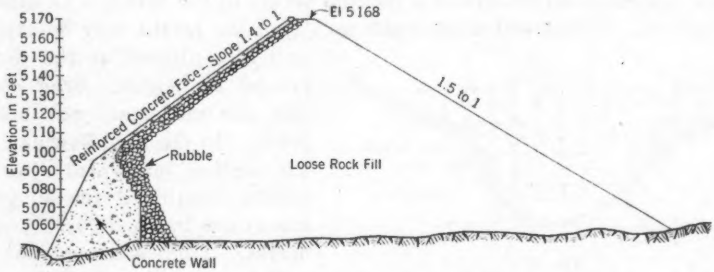

FIG. 4.—CROSS-SECTION OF BUCKS DAM

In all quarry work there is a certain amount of earth, spalls, and dust that results from the operations. Some of this waste should be rejected, but a certain quantity finds its way to the dump of the fill. It is necessary to wash this quarry waste into the fill and not allow it to accumulate in any one zone or position on the slope of the fill. At every place where the rock is dumped, there should be at least two standard fire streams of water under 100 lb pressure, constantly washing the fines into the rock. It is inevitable that the smaller rocks and most of the quarry waste will accumulate at the top of the dump. The necessary roadways require a surfacing of quarry waste. If the dam is built in vertical stages, the accumulated layer of small material at the top of any stage should be washed into the mass, but as this is rarely possible, it should be discarded. If left in place, such a layer of small material invites large local settlement, especially when the water load is applied.

It is realized that a heavier mass will result if fine material can be washed into the interstices of the fill. Such a course is advisable, only after the fill of rock has been made. The procedure should not result in a fill composed of rock in some places and masses of small material in others.

To the extent that it is possible in construction, the equipment for moving and placing the rock should be arranged to handle rock of about the same size should it be found necessary to open more than one quarry. Fills made in stages should have each stage fairly well complete before the next above is

placed. Usually, in large dams, work is carried on from both sides of the canyon, in which case there should not be any great difference in the height of the stages.

Settlement.—All rock-fill dams settle. The settlement is due to the crushing of the bearing points of the rocks under increased load or to the displacement of small rocks by large ones. If the fill is made as described, the point of one rock will rest, in general, upon a surface of the rock below. As pressure increases, the rock point will be crushed, and, finally, when the water pressure is on the dam, the maximum forces will be exerted and the rock points will be crushed until a balance is established between the compressive forces and the resistance of the rock.

Weather effects will operate to increase the disintegration of the rock and this is especially true in regions with cold winters. However, in the interior mass of a dam, the effects of weather are not so great as upon the surface.

The maximum settlement of a rock-fill occurs in the first few months after being placed. Settlement of as much as 5% of the height may be expected; and it continues as the dam increases in height. After completion the settlement continues for years. In the first Bowman Dam, the vertical settlement after completion continued for 50 yr and amounted to about 1.25% of the height. This settlement was quite uniform for about 20 yr with a total of 1 ft in 100 ft of height and then it was much more gradual.

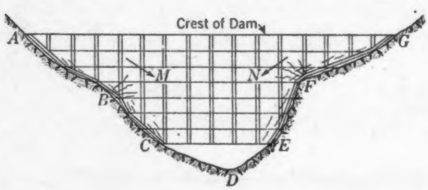

FIG. 5

Settlement in a dam is not uniform; nor is it vertical. Under usual conditions, the rock is dumped into the canyon from both sides, the fill advancing toward the center. There is a settlement of the rock toward the center as indicated by the arrows, *M* and *N*, in Fig. 5. This movement continues after the fill is complete. Fig. 5 indicates a typical profile across a canyon in which a dam may be built. The presence of a cliff, *E-F*, results in different settlements within a short distance. The configuration of the canyon wall determines to a large extent the difference in settlement.

Settlement due to the water load is an important factor in rock-fill dams. At the Dix Dam, the vertical settlement in the central portion 3 yr after completion amounted to 1.75 ft and the horizontal settlement was 1.50 ft, the height of the dam being 270 ft. This represented a movement approximately at right angles to the water-face of the dam which is mostly on a slope of 1 : 1. At the Strawberry Dam the settlement in the central section, 140 to 145 ft in height, was 1.62 ft vertical and 0.95 ft horizontal in a period of about 7 yr, indicating a movement at an angle of 75° to the slope of the water-face which is mostly 1 : 1. These measurements were taken at the top of the dam. The general indications are that the vertical settlement will continue a longer time than the horizontal, the latter being due to water pressure.

There are indications that, sometimes, movements will occur that are greater than those indicated above. This is especially true where the im-

pervious element is placed in the center of the dam. In this location it is subjected to full water pressure. In the Crane Valley Dam, with a core wall of concrete, a horizontal movement of several feet has occurred at the top.

In the Salt Springs Dam (see Fig. 6), provision was made for ultimate settlements of 6 ft vertically and 4.2 ft horizontally.

The character and location of the settlements of this rock-fill are important as determining the design of the impervious element of the dam. The movements from the side toward the center, indicated by arrows, *M* and *N* (Fig. 5) will result in closing the expansion joints in the central section and opening those on the flanks. Dams such as Dix, Strawberry, and Salt Springs, show this effect. Again, where differential settlement is possible as at Point *F*, Fig. 5, cracks may be formed in the concrete facing. In large dams the necessity of expansion joints where the facing joins the rigid rock of the canyon wall is also indicated.

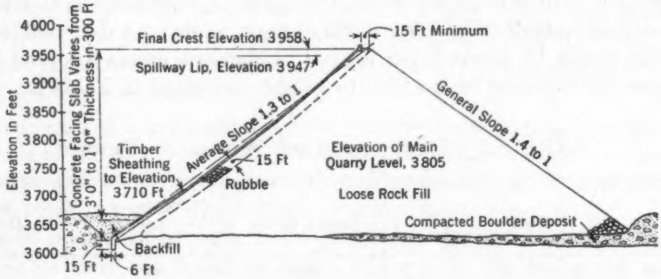

FIG. 6.—CROSS-SECTION OF SALT SPRINGS DAM IN CALIFORNIA

In the end, the depth of settlement and the question as to whether it is uniform throughout the mass of the rock-fill will be determined largely by the character of the fill, as discussed above. If layers of quarry spoil or gravel are permitted to accumulate in the rock-fill, excessive local settlements may be expected.

Cross-Section.—In the first rock-fill dams, the down-stream face was built on a slope of 1 : 1, or steeper. The up-stream slope was usually the same which resulted in a width of base of about twice the height. These steep slopes made it necessary to protect each face with rubble in order to retain the loose rock-fill.

Escondido Dam of 1895 may have been the first one to be built in which the down-stream face was composed of the unconfined loose rock. Probably Bear River and Meadow Lake Dams (Items 10 and 11, Table 1) were the last ones built with the down-stream face of rubble. In all the older dams there was a tendency to build the upper portion with steep slopes such as 0.75 : 1 and 0.5 : 1. The undoubted reason was to reduce the rock contents of the dam and the resulting cost.

There does not seem to be any good reason, except a possible saving in cost, for making the down-stream face other than the natural slope of rock dumped from cars. It is possible to have this slope as steep as 1 : 1 with large rocks only. However, in the ordinary construction of such dams, the rock varies in size and a loose rock-fill will generally assume a slope of 1.3 : 1 or

1.4 : 1 when the rocks are allowed to roll down the face of the fill. To make the slope steeper than 1.4 : 1 will usually require a second handling of the rock and there is no reason why this should be done.

The slope of the up-stream face is a subject upon which there is considerable difference of opinion. This is evidenced by the different slopes adopted by various engineers, as shown in Table 1. One of the controlling elements is found in the necessity of sufficient width of base to withstand the horizontal component of the water pressure. Experience with the Bowman Dam indicates that a base width equal to twice the height will stand without failure. Other dams noted in Table 1 have approximately the same ratios of base-to-height. However, in later dams, the ratios of width of base to the height have been increased, ranging from 2.5 : 1 up to 3 : 1. In low dams, the width of crest adds to the width of base a distance that is a factor in the total. This value decreases as the height increases. A crest width of 15 ft is a minimum, and usually a road is required across the dam. On a dam 100 ft high a 15-ft crest would add about 6% to the width of base, while on a dam 300 ft high the proportion would be about 2 per cent. The cross-sections adopted in recent dams have the ratios of base width to height, indicated in Table 2.

TABLE 2.—Ratio of Base Width to Height

No.	Dam	Height, in feet	Width of base, in feet	Ratio: Height to base
20	Dix	270	691	1 : 2.56
22	New Bowman	168	375	1 : 2.23
23	Bucks	118	340	1 : 2.90
24	Salt Springs	328	930	1 : 2.84

On an assumption of a dam 200 ft high, with a crest 15 ft wide, a down-stream slope of 1.4 : 1 and a unit weight of 100 lb per cu ft for the loose rock and with the water load, ratios of height to base of 1 : 2.25; 1 : 2.5, and 1 : 3 would have sliding factors (ratios of weight of rock to water pressure) of 4.50, 5.14, and 6 : 45, respectively. These ratios are practically constant for all heights of dams. Within the limits of the ratio of height to base width, mentioned, all such dams have ample safety against sliding. It is possible, therefore, to adopt any slope for the up-stream face that may be desired.

In Table 1 the principal face slopes are given for a number of dams. The writer believes that the slope should not be less than 0.75 : 1 and that a slope of 0.5 : 1 is too steep. Much depends upon the amount of rubble rock behind the facing. In the Relief Dam, 140 ft high, the rubble section is 100 ft thick at the bottom and 13 ft thick at the top. Such a section is stable in itself, and the slope of 0.75 : 1 on the up-stream face is ample. However, the cost of the rubble indicated that a flatter slope and less rubble would have been more economical.

In any slope steeper than the natural slope of the loose rock, it is necessary to carry up this rubble facing with the loose rock-fill in order to retain the latter. Under these conditions it is necessary to maintain the derricks by which the

rubble is placed, upon the face of the rubble already laid, usually a difficult operation.

In the case of the Salt Springs Dam it was decided after discussion, to make the up-stream slope of the loose rock-fill 1.3 : 1, or what was practically the natural slope (see Fig. 7). The principal reasons for this conclusion are as follows:

(1) The loose rock-fill could be built in advance of the rubble facing. To a certain extent this eliminated danger to workmen from rocks rolling down the slope. However, the controlling motive was the fact that time was given for the loose rock to settle. As settlement is a function of time and of superincumbent weight, building up the loose rock ahead of the rubble permitted the fill to take its initial settlement before the rubble was placed upon it. The settlement is usually considerably greater in the first three or four months than thereafter.

(2) The design of the rubble facing permitted the use of caterpillar derricks resting upon the rubble as it was built up. Rock for the rubble was brought in trucks and dumped down the loose rock-fill at convenient points, the caterpillar derricks moving about as required.

It is recognized that a steep slope for the up-stream face reduces the quantity of all parts of the dam—loose rock, rubble and facing—but the aforementioned advantages are believed to outweigh the increased cost of the flatter slope. This is especially true of the element of initial settlement.

It is the common custom to make the up-stream slope a curve concave to the water. This prevents buckling of the concrete facing under the settlement. Usually, the result is obtained by varying the slope. At the Strawberry Dam, several slopes, varying from 1.2 : 1 at the bottom to 1 : 1 at the top, were used. At the Salt Springs Dam, at the central section, the up-stream face was made a curve, concave to the water. Measured on a horizontal line from the theoretical slope of 1.3 : 1 (the rock at the crest elevation) the offset was 13.6 ft up stream, and at an elevation of 200 ft below, the offset was 10.5 ft down stream from the theoretical slope. In plan, the Salt Springs Dam has a crest slightly curved up stream. The face is a warped surface.

Summarizing the elements that control the design of the cross-section, the down-stream slope may be made of dumped rock, unconfined, 1.3 : 1 or 1.4 : 1; the crest width should be at least 15 ft; and the up-stream face should preferably be the natural slope, which can be made 1.3 : 1. This will result in a stable dam with base width 2.7 times the height, to which is added the crest width. To prevent buckling of the facing, the profile of the up-stream face should be made convex to the water.

Rubble Wall.—By this term is meant the layer of rock interposed between the impervious facing of the dam and the loose rock of the main fill. It is variously referred to as hand-laid or packed rock, derrick-laid rock, placed rock, etc. The term, rubble, may be open to criticism.

The function of the rubble is to act as a semi-rigid member between the rigid facing and the loose fill which is subject to settlement in several directions. The rubble also helps to bridge inequalities in the loose rock and to furnish a uniform bearing to the facing.

The rubble wall is an important part of the dam. It should be built of large rocks which are placed in positions in bearing contact with each other. The remaining spaces should be carefully filled with small rock and spalls wedged together to form a compact mass. The rock may be laid in horizontal planes, but made interlocking between layers by projecting rocks. Large rectangular rocks on the outer surface should be placed with faces roughly parallel with the slope.

If it is well built, a rubble wall is capable of resisting distortion or overturning stresses comparable to masonry walls. Note is made of well-built rubble retaining walls on the Southern Pacific Railroad where the line crosses the Sierra Nevada. A wall at the summit of the mountains, with a batter about 0.25 : 1 has carried, for many years, a double-track railroad upon which is operated the heaviest locomotives. Other instances will occur to the reader.

Table 1 gives the thickness of rubble in different dams. With the exception of Morena and Relief Dams, there is a rough correspondence in the dimensions in that the thickest part is at the bottom which narrows to a nominal thickness at the top. The idea seems to have been to vary the thickness in accordance with water pressure. In some dams the rubble was made of only nominal thickness as in the Beaver Park Dam. The thickness given for the first Bowman Dam is incorrect as the rubble facing was found to be only 2 to 3 ft thick when the dam was demolished.

At Salt Springs (Fig. 8) the writer believed that there were two controlling factors in determining the thickness. Water pressure indicated a maximum at the bottom. However, settlement of the loose rock-fill, known to be unequal, is a maximum at the crest of the dam. This factor required a considerable thickness at the crest. As built, the thickness of the rubble was made uniform from top to bottom, being 15 ft normal to the face, or about 25 ft wide on the horizontal. A similar design was followed in the Cucharas Dam where the rubble was made 30 ft thick normal to the face slope of 1 : 1.

An argument in favor of the uniform thickness of rubble of reasonable dimensions is that the derricks used to lay the rock can operate upon the wall itself without change from base to crest, which is an important item in reducing cost (see Fig. 9).

In the final analysis, the thickness of the rubble is a matter of judgment. Experience indicates that the dams built in recent years have been provided with rubble walls of sufficient thickness to perform the necessary function of distributing the water load to the loose rock-fill and of resisting excessive distortion under unequal settlement.

Cut-off Wall.—The cut-off wall is an important element in rock-fill dams, as upon it depends the connection of the facing with the bottom and sides of the canyon walls. It should be of dense concrete, wider than the facing if of concrete; and the depth into the rock of the canyon should be sufficient to cut off all visible seams in the rock and also to provide resistance to the grout injected under pressure. Grout holes should be drilled through the concrete and should penetrate deeply enough into the rock so that grout under a pressure of at least 100 lb per sq in. may be forced into any seams not closed by the wall. The design and dimensions will be determined by each locality.

FIG. 7.—GENERAL VIEW OF SALT SPRINGS DAM, WITH SPILLWAY ON LEFT BANK

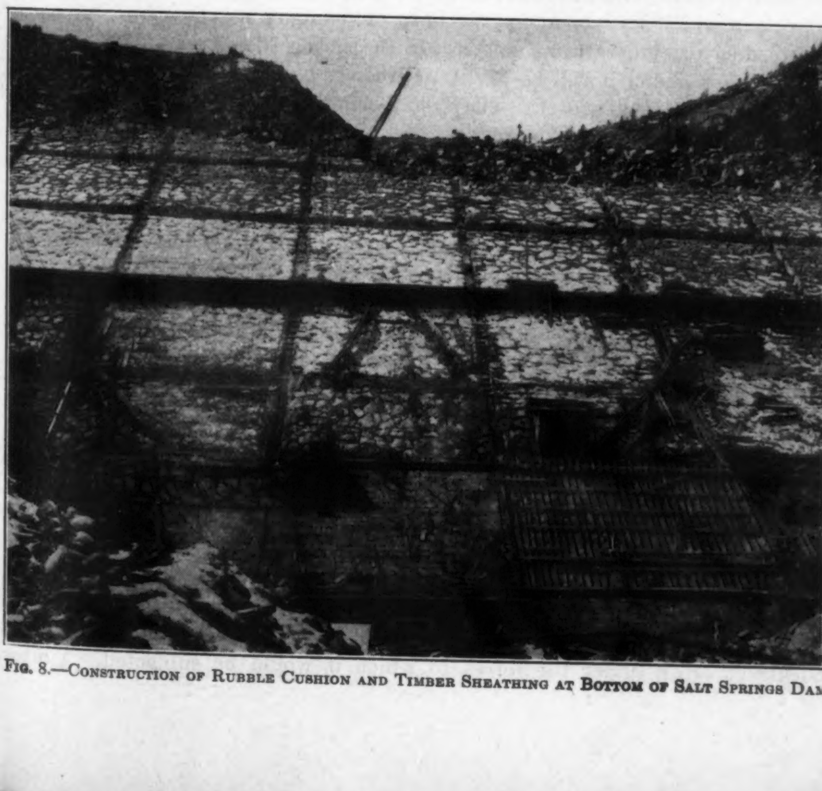

FIG. 8.—CONSTRUCTION OF RUBBLE CUSHION AND TIMBER SHEATHING AT BOTTOM OF SALT SPRINGS DAM

Facing.—Dams of the type under review must be provided with an impervious element to resist the water. As previously mentioned, the facing is the first and only resistance to the water as the loose rock and rubble offer practically no obstruction to the flow.

The position of the impervious element is important. A few dams have been built with the steel or concrete diaphragm in the center of the dam. The most conspicuous example of the method was the Lower Otay Dam. There are a number of objections to placing the impervious element in the body of the dam. All the rock-fill up stream from the diaphragm is submerged, and its function of resisting the forces of the water is nullified by partial flotation of the rock and by the fact that the water pressure is exerted where the up-stream portion of the rock cannot resist it. Again, settlement of the loose rock is not vertical but approximately along a resultant of the forces of gravity and of the water. In settling, the rock distorts the diaphragm and possibly ruptures it, and, as it is inaccessible, inspection and repairs are not possible. Other disadvantages may be mentioned, but those given should be sufficient to cause the rejection of this type of design.

It is believed that the only proper place for the impervious element is on the water-face of the dam, directly in contact with the water. In such a design all the rock acts to resist the water, the facing is open for inspection and repair, and its construction can follow as far behind the rock-fill as possible.

In the design of the facing of a rock-fill dam of medium or large size the engineer is confronted with the problem of uniting a fairly rigid facing to a mass of rock in which settlement is expected. The facing is exposed to changes of length due to temperature changes. In the spring the reservoir will be filled. In the late summer it will be partly or entirely emptied, which condition will prevail through the winter. The facing will then be exposed to ranges of temperature of 100° F, or more. In dams the facing may have a horizontal length from 500 ft to 1 000 ft, or more, and a width on the slope of from 200 ft to 500 ft. If made of concrete or steel and if it is free to move, the facing may expand and contract from 2 to 10 in. under the conditions named. However, movement cannot take place where the facing joins the walls of the canyon, which do not move.

The first factor in the problem (that of settlement which may be of the order of several feet) would indicate that the facing should be separated from the rock mass and left free to slide on the rock mass as it settles. The second factor, annual changes due to temperature would call for movements which, it is believed, cannot be permitted. Horizontal movements would take place, but in any possible plane of sliding the vertical movements, or rather movements along the inclined face of the dam, would not take place. Under low temperatures the facing (if made of steel or concrete) would contract and the movement would be downward. With rising temperatures, the movement would tend to slide the facing upward which would be resisted by the weight of the facing, friction on the sliding plane, and resistance where the facing joins the side walls of the canyon. It is believed that in any practicable case the vertical movement would not take place and that the facing, if made of steel or concrete, would buckle or crush under the forces to which it would be subjected. Timber

facing, which is adapted only to relatively low dams, is sufficiently flexible to adjust itself to the changes due to settlement or to temperature changes.

It should be noted that the rubble cushion behind the facing, if properly constructed, forms a semi-rigid rock mass that resists, to a large extent, the deformation due to settlement. The differential movements between the rock-fill and the rubble wall with the facing resting upon it, is believed to take place in that zone of contact rather than on the plane between the facing and the rubble.

FIG. 9.—CONSTRUCTION OF RUBBLE CUSHION, SALT SPRINGS DAM

The plan generally followed is to fasten a steel or timber facing directly to the rock of the rubble wall. Taking account of the various factors involved and basing the opinion on experience, the writer believes that the best method is to pour a concrete facing directly upon the rubble wall, taking care that interstices are left in the rock work into which the concrete is poured in order to secure a firm bond between the two elements. In addition, grooves or chases should be built in the rubble at the joints in the concrete facing to support the slabs over the joints, thus giving a closer bond. In this manner the facing is secured to the rubble and temperature changes are resisted. In settlement, the facing follows the rubble and stresses caused by local settlement can be resisted by the reinforcement. It is possible that the facing may be crushed in places, but experience shows that this does not often occur. Reference is again made to Bucks Dam (see Figs. 3 and 10) where a concrete facing more than 1 000 ft long, poured directly on the rubble and containing no expansion joints, has not cracked in the eight years since it was constructed. At Salt Springs (see Fig. 11), the facing was poured directly on the rubble and although some minor crushing has taken place, repairs are easily made at low water. On the contrary, at the

Strawberry Dam, the design provided for a complete separation between the two elements in the upper portion of the facing, about 100 ft wide along the slope. The rubble was made smooth by a covering of cement mortar and roofing paper. The facing is in good condition but the expansion joints have opened wide at the ends of the dam and closed together at the center.

It must be recognized that these conclusions have been reached after consideration of the several contradictory factors that enter into the problem of constructing an impervious face of rigid concrete upon a rock-fill subject to continued settlement and to temperature changes.

The problem of a dam with a sheet steel facing is similar to that with a concrete face. In the two rock-fill dams listed in Table 1, with steel facing (Items 28 and 29), the steel plates were fastened by bolts to the rubble wall and were provided with expansion joints. In the highest of the two dams, the Penrose-Rosemont Dam in Colorado, the dry rubble is faced with rubble masonry laid in cement mortar 4 ft to 2 ft thick, with a plaster face next to the steel facing.

Timber Facing.—With the exception of the small Chatsworth Dam (Item 6, Table 1), the facing of the older dams was made of timber. Timber sills, 12 by 12 in., or larger, or made of hewn timbers, are buried in the rubble wall on the face of the dam. It was common practice to space them 6 ft, center to center, the sill being in vertical planes at right angles to the water-face. Such sills were fastened to the rock with large bolts. In the lower half of the facing, three layers of 2-in. or 3-in. planks, made as wide as available, were spiked to the sills, the central layer being placed at right angles to the other two. In the upper section, two layers of planks were used. Small spaces between the sheathing and the rubble were carefully filled with spalls to give as much support as possible to the facing. All joints were caulked with oakum. The result was a good water-tight facing that lasted, with minor repairs, for at least 20 yr. Some of the timbers of the first Bowman Dam lasted 50 yr. On the Sabrina Dam the timber face lasted about 20 yr, after which it was replaced by another timber face in 1929. In remote locations, where concrete is costly and timber plentiful it is good practice to use timber facing. It has the advantage of cheapness, which is often an important factor in new enterprises. Moreover, it is flexible, and its use at first permits the rock-fill to take most of its settlement before a final facing of concrete is applied.

Concrete Facing.—On the higher dams, built in recent years, the common practice has been to use reinforced concrete for facing. The small dam at Chatswork Park, in Southern California, built in 1896, was probably the first dam in which concrete was used as a facing. The Relief Dam (1907–1910) is believed to be the first of the larger dams in which reinforced concrete was used.

There was a question as to whether the concrete would be water-tight, but it was there proved that concrete would be practically water-tight under pressures up to 140-ft. Experience at Dix and Salt Springs Dams indicates that water pressures up to 300 ft can be resisted safely by concrete. There is hardly any way in which the thickness can be determined. At Dix Dam the slab is 18 in. thick under a head of about 260 ft. At Salt Springs, the thickness was made 3 ft, under a 300-ft head. The element of settlement and cracking of the concrete

Fig. 10.—Construction of Monolithic Concrete Face, Bucks Dam, in California

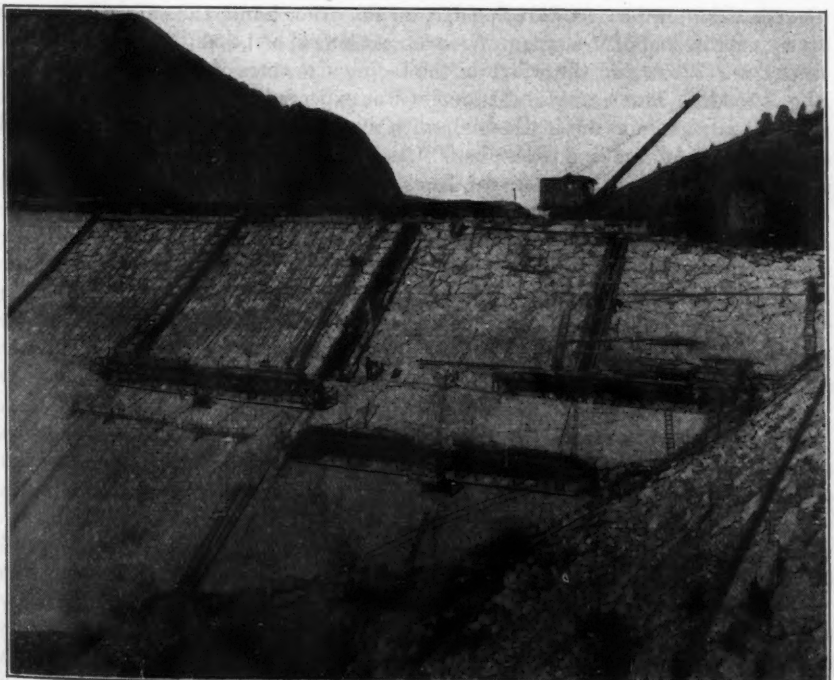

Fig. 11.—Construction of Concrete Facing, Salt Springs Dam

seems to indicate a substantial thickness of slab. It is believed that a slab thickness of 1% of the head of water will provide ample thickness. It is also believed that the slab should not be less than 12 in. thick at the top and preferably thicker where climatic conditions may lead to disintegration from frost.

At about the time the Relief Dam was under construction, a concrete facing was projected for the Morena Dam. The rubble backing was set in cement mortar which was found to be so water-tight that the concrete slab was only installed in the lower sections of the dam.

Concrete facing of rock-fill dams has usually been placed in slabs ranging from 50 to 60 ft square. Expansion joints permit adjustment under contraction and settlement of the fill. However, in the Bucks Storage Dam, the concrete facing was built without expansion joints and the reinforcement was extended across the construction joints so that the slab is practically monolithic. It may seem inconsistent to have approved two different forms of facing in two important dams being built at the same time, as was done by the writer at Bucks and at Salt Springs. However, the conditions are different. At Bucks, owing to the construction of a gravity concrete dam across the main stream as a part of the structure, the maximum width of the concrete facing measured up and down the slope is about 127 ft, while the crest length is 1 200 ft. In plan, the dam has two angles dividing it into three sections, the angles being convex to the water. Construction joints were spaced 60 ft apart horizontally and 31 ft 10 in. apart up and down the slope. The conditions were believed to warrant the omission of expansion joints. At Salt Springs, on the other hand, the canyon was relatively narrow and of V-shape. With a crest length of 1 300 ft, the height at the center was 328 ft and the width of the facing slab, measured on the slope, was about 550 ft. Much greater settlement was expected than at Bucks, and it was deemed advisable to build the slab with expansion joints, which were placed on 60-ft centers, making the slabs 60 ft square.

Reinforcement of the slab of Bucks Dam consisted of one layer of deformed bars, in the center of the slab. The vertical bars were 1 in. square spaced 12 in. center to center, and $\frac{3}{4}$ in. square bars spaced 9 in. apart, the latter being used in the upper reaches of the slab. The horizontal bars were $1\frac{1}{4}$ in. square in the lower reaches, spaced 11 in. to 12 in. apart, and 1 in. square in the upper reaches, spaced 9 in. and 11 in. apart, the bars being about 0.75% of the slab area. Reinforcement of the slab of Salt Springs Dam was made with one layer of bars in the center of the slab down to an elevation 184 ft below the spillway lip, where the slab was 27 in. thick, and two layers below that elevation. Bars are 1 in. square and 1 in. round, spaced from 3 in. to 14.5 in., center to center, depending upon position. The reinforcement was the same in both directions and did not pass either the construction joints or the expansion joints. In general, the area of bars was made about 0.5% of the area of the slab.

An exact calculation of the percentage of reinforcement is not possible although the stress can be estimated under temperature changes. The concrete of the slab is poured directly upon the rubble wall that supports it. The concrete enters into the interstices of the rock, fills the inequalities and adheres to the rock. The concrete is thus held in position, and expansion or contraction is prevented. It might be questioned whether the concrete should not be formed

so as to be separated from the rock in order to permit movement when the settlement of the loose rock-fill takes place. There does not seem to be any valid reason why this should be done and there are a number of reasons why it should not. Experience has shown that concrete slabs formed on well-laid rubble do not crack from expansion and contraction of the concrete. Some settlement cracks are to be expected. However, the slab of Buck Dam (elevation more than 5 000 ft) has passed through eight winters with temperatures below zero and does not show any cracks. The leakage is negligible. The experience at other dams such as Relief or Strawberry is the same. The rubble masonry face of Morena Dam, which has been in service more than 20 yr (see Item 15, Table 1), has proved ample for the purpose.

Steel Facing.—This type of facing has been used on two dams in Colorado with success. The Skagway Dam built in 1901 is 70 ft high with the up-stream face on an angle of 30° from a vertical. The crest is 405 ft long. Steel plates 5 ft wide and 15 ft long and varying in thickness from 0.5 in. at the bottom to 0.25 in. at the top. Horizontal joints were made with butt straps and the vertical joints were made with 5 by 4-in. angles, with 5-in. legs outstanding; 2-in. by $\frac{3}{8}$-in. filler bars were riveted between the toes of the adjacent angles, temperature adjustment taking place by the flexure of the angles. The dam has been in use 35 yr (see Item 28, Table 1). The facing has performed its function, and the damage due to corrosion was found, in 1932, to be negligible.

The Penrose-Rosemont Dam, built in 1932, is 100 ft high, 580 ft long on the crest, and was built with a water-face on a slope of 0.5 : 1. The steel plates are 8.5 ft wide by 20 ft long, varying in thickness from $\frac{3}{8}$ in. at the bottom to $\frac{1}{4}$ in. at the top. Horizontal joints were made by lapping the plates, bolting them together every 18 in., and welding. Vertically, two plates were connected by a T, making a plate about 40 ft long. These lengths were connected by a contraction joint made of plate steel rolled to a semi-circular form. The plate facing was fastened to the rubble with bolts. Steel facing is entirely practicable and may have an increasing use in the future.

Construction and Expansion Joints.—The difference between the two types of joints is one of function. Construction joints representing the boundary of a concrete slab poured at one time have the concrete of the next pour directly in contact with the first. At Bucks Dam both horizontal and vertical joints were construction joints. In practically all cases, all horizontal joints become construction joints as such joints, being always in compression, never tend to open although they may be subjected to shear. Vertical joints, on the contrary, are subject to movements of the mass of the dam in which case they may either open or close. As the movements of the rock-fill under settlement are toward the center of the dam from the flanks, the joints in the central portion will tend to close and those at the abutments will open. Where there is an abrupt change in the profile of the canyon wall, as at Point *F* in Fig. 1, the joints will open. For these reasons it is advisable to build the expansion joints at the center wide open and those at the abutments closed.

In dams of some height there is a tendency for cracks to appear where the concrete facing joins the canyon walls at the cut-off wall. These cracks are due to settlement and the extent of the cracking depends upon the care used in

placing the rock-fill and the rubble. At the line of contact between the concrete face and the concrete cut-off wall it is necessary to connect the two with reinforcing bars. Movement of the facing under settlement will cause bending stresses at the junction line. This movement will not all take place at one line, but there will be a zone along the contact line where the movement changes from nothing at the cut-off wall to a maximum in the body of the dam. Provision should be made in such cases for additional expansion joints which will permit motion without rupture of the concrete.

When the pressure is not great construction joints may be made in the concrete without seals, offsets being made to form keys. Such joints will generally be kept tight if the reinforcement is carried across the joint. However, it is common practice in important dams to provide some form of seal, both in construction and in expansion joints. In the latter, where movement is expected which will cause the joint to open or close, there must be a seal against the passage of water. The designs take various forms. In timber facing no such joints are necessary as there is in effect a joint at each board of the facing. In concrete, the joint usually consists of sheet copper extending into the adjoining slabs with a U-shaped section covering the open joint. The open part

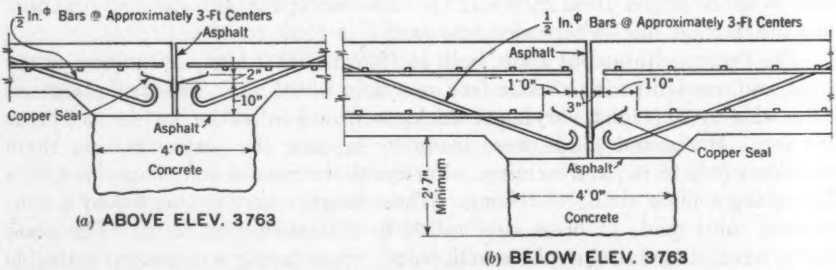

FIG. 12.—VERTICAL EXPANSION JOINTS, SALT SPRINGS DAM

of the joint is usually filled with asphalt. At Salt Springs (see Fig. 12) the sheet copper was 33 in. wide, the sheets being brazed together at junctions. In steel facings, the expansion joints may be made in different ways, as illustrated in the two Colorado dams cited.

Other elements of rock-fill dams, such as outlets, valves, spillways, etc., that are similar to other types of dams, do not require discussion herein. However, it should be borne in mind that a rock-fill dam may be destroyed by the passage of a flood over the crest, as in the case of the Lower Otay Dam, where a masonry dam would not have been injured. This points to the necessity of ample spillway capacity which should be free from all gates, flash-board structures, and such other construction that might cause the spillway to clog at the time of flood.

CONCLUSIONS

The paper is limited to the design of rock-fill dams. A certain amount of historical and statistical data is included because the type has been the result of

evolution extending over a course of years. Rock-fill dams are not the most stable type, but there are a number of reasons for their adoption.

A rock-fill dam is defined as one made up of loose rock forming the mass of the dam, an impervious face next to the water that may be made of timber, steel, or concrete, and a mass of rubble placed between the loose rock and the face. Dams made partly of rock and partly of earth are not included.

The foundation should preferably be on solid rock, but the dam may be partly on compacted material, provided that the impervious face is connected to a cut-off wall on the bottom and sides of the canyon that will effectually shut off water.

The material composing the body of the dam should be a hard, solid rock, free from joints that would permit the rock to fracture under load, and where the crystalline structure has not been broken down by earth stresses. The material must not weather or be broken by freezing. Granite and limestone have been used, but no examples are known of the use of sandstone. The rock material must be placed in the dam in a way such that rock bears against rock. Fine material may be sluiced between the rocks of the mass after they have been placed, but not before or in such manner that rocks will rest upon the fines.

Dams have stood where the base was little more than twice the vertical height, but the section should be such that the ratio is greater. The downstream slope should be that of free falling rock, generally about 1.30 : 1 to 1.40 : 1. The slope of the water-face may range from 0.75 : 1 to 1.30 : 1, but the latter slope is believed the best for constructional and settlement purposes. The sliding factors will then be ample.

In plan, the dam should be somewhat convex to the water and the up-stream profile in section should also be a curve that is convex to the water.

The rubble cushion between the rock-fill and the impervious face should be of uniform thickness throughout its height. The thickness of the rubble is a matter of judgment and Table 1 gives the data for several dams. It is believed that a minimum thickness of 15 ft is necessary, and a greater thickness is advisable. The rubble should be carefully laid and packed with spalls.

The facing may be of timber, concrete, or steel. Timber of two and three layers of plank fastened to sleepers built in the rubble is sufficient for dams of moderate height. Concrete should be reinforced and of a thickness at least 1% of the height, or more. On low dams of considerable length, with the dam convex to the water in plan, the concrete may be laid without expansion joints, provided it is reinforced against temperature changes. In dams in narrow canyons and of considerable height, the facing should be divided into squares of approximately 50 ft on a side, by expansion joints and construction joints. Grooves or chases should be formed in the rubble and filled with concrete upon which the joint may rest. Copper seals should be used and the joints filled with asphaltum. All rock-fill dams settle over a long period of years, although the major settlement takes place in the first years. Settlement is roughly at right angles to the water-face and also along the axis of the dam from the canyon walls toward the center. It is not possible to construct a concrete face that will move independently of the settlement of the rock mass. The concrete should be poured directly on the rubble face and attached thereto by filling the openings in

the rubble with concrete. On account of axial settlement, expansion joints at the central section should be built wide open and those at the ends, closed. Steel facing, made of plates and shapes, forms an excellent and durable facing which will fulfill all the functions of the other methods. Experience with steel facing has been limited to two dams, but is believed to be satisfactory.

Other details of design are not discussed because they are the same nature as those of other types. One element should be emphasized—on account of the danger of destruction if the dam is overtopped with water, the spillway should be of ample proportions and left entirely free from all obstructions, such as flash-boards, gates, etc.

The design of rock-fill dams is a matter of judgment, based upon experience. Opinions will differ, and the writer hopes that discussion will be directed to matters of design, that being the subject-matter of this paper.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

DESIGN OF REINFORCED CONCRETE IN TORSION

By Paul Andersen,[1] Assoc. M. Am. Soc. C. E.

Synopsis

In analyzing space structures torsional moments may be readily determined by the method of successive approximations.[2] For a given torsional moment an economical section is the square. If it is reinforced with 45° spirals such a section lends itself to mathematical investigation and a rational design procedure can be developed. The percentage of reinforcement in a rectangular section can also be approximated. For convenience of reference, a complete list of the letter symbols used in this paper is given in the Appendix.

Distribution of Torsional Moments

That the method of moment distribution can be applied to space structures[2] has been demonstrated by Hardy Cross, and N. D. Morgan, Members, Am. Soc. C. E. In order to find torsional moments in a three-dimensional frame it is necessary to supplement the "flexural" beam constants with "torsional" beam constants. Torsional stiffness is the moment which, applied at one end of a member, will produce unit rotation at this end, while the other end is fixed. It follows that the torsional stiffness of a member of length, L, is:

$$K_T = \frac{E_s T}{L} \qquad (1)$$

in which E_s = the modulus of elasticity in shear; and T = the torsion factor which, for a circular section, is the polar moment of inertia and for a rectangular section, approximately:

$$T = \frac{b^3 d^3}{3.58 (b^2 + d^2)} \qquad (2)$$

b being the width, and d, the depth of the member. The torsional carry-over factor is the ratio of $\frac{M_1}{M}$, in which M is an arbitrary moment applied at one end

Note.—Discussion on this paper will be closed in February, 1938, *Proceedings*.

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[2] "Continuous Frames of Reinforced Concrete," by Hardy Cross and N. D. Morgan, Members, Am. Soc. C. E., John Wiley & Sons, N. Y., 1932, p. 117.

of the member, which is assumed as free to rotate, and M_1 is the resisting moment at the other end of the member, which is assumed as fixed against rotation. These two moments, of course, are always equal, and, therefore, the torsional carry-over factor is always unity.

Fixed end torsional moments are directly proportional to the distance from the point of application of the external moment to the opposite support. Thus, for a beam with a constant moment of inertia, the fixed end torsional moments are,

$$M_f = M \frac{n}{m + n} \qquad (3a)$$

and

$$M_t = M \frac{m}{m + n} \qquad (3b)$$

in which m and n are the distances from the applied moment to the fixed ends, respectively. If members subjected to torsion join members in flexure, the flexural stiffnesses of the latter are expressed by,

$$K_F = \frac{4\,E\,I}{L} \qquad (4)$$

If

$$E_s = \frac{E}{2.25} \qquad (5)$$

is substituted in Equation (1), the corresponding torsional stiffness of a member becomes,

$$K_T = \frac{E\,T}{2.25\,L} = E \times \frac{1}{L} \times \frac{b^3\,d^3}{2.25 \times 3.58 \times (d^2+b^2)} \qquad (6)$$

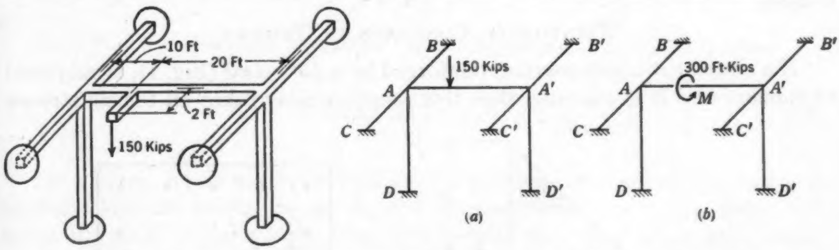
If the relative stiffness of a member in flexure is expressed by $\frac{I}{L}$, the relative torsional stiffness will be $0.031 \times \frac{I}{L} \times \frac{b^3\,d^3}{b^2 + d^2}$.

The example in Table 1 will illustrate the procedure for distributing moments in a space structure due to an eccentrically loaded beam. The problem is conveniently solved by considering first the effects of the direct load on the center beam as in Table 1(a), and then the moments due to the torsional moment as in Table 1(b). For the sake of simplicity, side-sway is neglected even if there is some side-sway present, due to the eccentric load of 150 kips in Example (a), Table 1, deflecting Points A and A'. It is customary, in plane structures, to stop the process of moment distribution after a distribution, since the sum of the moments around a joint should equal zero. However, in a space structure this will not make the torsional moments in the members balance and the torsional moments (external and internal) in any one member should balance. It is just as correct, of course, to stop after having distributed the flexural moments as after having balanced the torsional moments. If the process is carried to the proper number of distributions it may be stopped after either, as the error would be insignificant.

It should be noted that the flexural and torsional moments in Table 1 can also be found by the slope-deflection method. The unknowns will be the

rotations of Points A and A' in the two perpendicular planes; this makes a total of four rotations, which can be determined from the four equations expressing the condition that the two sets of moments at each joint, A and A', must balance.

TABLE 1.—MOMENT DISTRIBUTION IN SPACE STRUCTURES



Description	AB	AD	AC	AM	$A'M'$	$A'C'$	$A'D'$	$A'B'$	AB	AD	AC	AM	$A'M'$	$A'C'$	$A'D'$	$A'B'$
	(a) EFFECT OF DIRECT LOAD ON THE CENTER BEAM								(b) MOMENTS DUE TO THE TORSIONAL MOMENT							
Type of stress*	T	F	T	F	F	T	F	T	F	F	F	T	T	F	F	F
Stiffness ratio..	0.2	0.3	0.2	0.3	0.3	0.2	0.3	0.2	0.3	0.3	0.3	0.1	0.1	0.3	0.3	0.3
Carry-over factor.......	1.0	0.5	1.0	0.5	0.5	1.0	0.5	1.0	0.5	0.5	0.5	1.0	1.0	0.5	0.5	0.5
	MOMENT DISTRIBUTION, IN FOOT-KIPS															
Fixed-end moments.......	..	..	..	167	−83	..	..	..	..	..	..	200	−100	..	..	..
First distribution.........	−33	−50	−33	−50	25	6	25	16	−60	−60	−60	−20	10	30	30	30
				12	−25							10	−20			
Second distribution.......	−2	−4	−2	−4	8	5	8	5	−3	−3	−3	−1	2	6	6	6
Final moments.	−35	−54	−35	125	−75	21	33	21	−63	−63	−63	189	−108	36	36	36

* T = torsion; F = flexure.

SHEARING STRESSES

The maximum unit torsional shearing stress in a rectangular section subjected to a torsional moment, M, is approximately,

$$v_m = \frac{M}{b\,d^2}\left(3 + \frac{1.8 \times d}{b}\right) \quad \text{.......................(7)}$$

For a given cross-sectional area this stress (which occurs at the middle of the long side, d) is a minimum when $b = d$; in this case it can be expressed by,

$$v_m = \frac{24\,M}{5\,b^3} \quad \text{.............................(8)}$$

For a rectangular section, the maximum unit vertical shearing stress due to a total vertical shear, V, equals,

$$v_m' = \frac{3}{2} \times \frac{V}{b\,d} \quad \text{.............................(9)}$$

and is dependent on the area of the rectangle.

As a rule, the bending moments that accompany heavy shears and torsional moments are comparatively small. The designer of concrete in torsion, therefore, is interested primarily in square sections or in sections that are nearly square. It should be noted that, in some cases, the beam that has a width greater than its depth may be still more economical than the square, because its maximum torsional shearing stress occurs where the vertical shearing stress is zero.

Reinforced Concrete in Torsion

If a circular concrete section, reinforced by a 45° spiral (Fig. 1), is subjected to torsion, and it is assumed that this reinforcement takes all tensile stresses

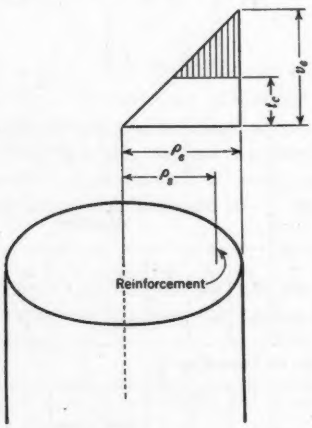


Fig. 1

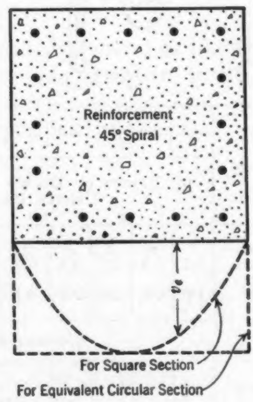


Fig. 2

greater than the permissible tension for plain concrete, it can be shown that the following relation exists[3]:

$$N A_s = (v_e - s_c)^2 (3\, v_e^2 + 2\, v_e\, s_c + s_c^2)\, \frac{\pi \rho_e^3 \sqrt{2}}{12\, v_e^3\, \rho_s\, s_s} \quad \ldots\ldots\ldots (10)$$

and for a square section (Fig. 2),

$$F\, N\, A_s = (v_e - s_c)^2 (3\, v_e^2 + 2\, v_e\, s_c + s_c^2)\, \frac{\pi \rho_e^3 \sqrt{2}}{12\, v_e^3\, \rho_s\, s_s} \quad \ldots\ldots\ldots (11)$$

in which, N = number of bars, 45° to the axis, cut by a horizontal plane; v_e = shearing stress along the edge of a cylinder, produced by a twisting moment, M; s_c = permissible unit tensile stress for concrete; ρ_e = radial distance in a horizontal cutting plane, measured from the axis of a cylinder to the edge; ρ_s = radial distance to the steel reinforcement; s_s = permissible unit tensile stress for steel; and F = a reinforcement efficiency coefficient relating to the variation in tensile stress along the edge. The coefficient, F, is always less than one and will approach $F = \frac{2}{3}$ as the pitch of the spiral decreases, this being the ratio between the area of the parabola representing the variation of

[3] "Experiments with Concrete in Torsion," by Paul Andersen, *Transactions*, Am. Soc. C. E., Vol. 100 (1935), p. 949.

shearing stresses along the edge and the area of the rectangle expressing uniform tensile stress in all the steel rods of the spiral.

If c is the perimeter of the spiral and p the pitch, it is seen that,

$$N = \frac{c}{p\sqrt{2}} = \frac{8\,\rho_s}{p\sqrt{2}} \qquad (12)$$

With $F = \frac{2}{3}$, substitute Equation (12) in Equation (11) and transpose:

$$\frac{A_s}{p} = (v_e - s_c)^2(3\,v_e^2 + 2\,v_e\,s_c + s_c^2)\,\frac{\pi\rho_e^3}{32\,v_e^3\rho_s^2 s_s} \qquad (13)$$

For a given square subjected to a known torsional moment all quantities in Equation (13) are known except A_s, the cross-sectional area of one spiral rod, and p, the pitch of the spiral. The designer has the choice of selecting either one of these unknowns and then to find the corresponding value of the other.

The design of a reinforced concrete beam in torsion is shown in Fig. 3, in which the main reinforcement consists of five 1-in. round bars, top and bottom; the vertical stirrups, $\frac{1}{2}$-in. round bars; and the spirals, $\frac{1}{4}$-in. round bars. The maximum torsional moment in the beam occurs when the eccentric load on the beam is close to the column; the moment distribution for this case is indicated in Table 2. It should be noted that whereas the maximum vertical shear along the beam varies according to a curved line the maximum torsional moment will vary as a straight line.

The maximum vertical shearing stress is $v_m' = \frac{60\,000}{\frac{7}{8} \times 24 \times 21} = 136$ lb per sq in.; and the maximum torsional shearing stress is, $v_m = \frac{3 \times 400\,000}{5 \times 12^3} = 139$ lb per sq in.

The beam should be designed first for vertical shear, adopting a lower allowable stress in the concrete, and then for torsion. Suppose that the total allowable shearing stress for concrete without web reinforcement is 90 lb per sq in.; it is now assumed that this stress can be divided into two parts in proportion to the magnitudes of the vertical and torsional shearing stresses: Of the vertical shear the concrete takes $\frac{136}{275} \times 90 = 44$ lb per sq in.; and, of the torsional shear, it takes $\frac{139}{275} \times 90 = 46$ lb per sq in. Applying Equation (13) to the part of the beam adjacent to the column support gives,

$$\frac{A_s}{p} = (139\text{–}46)^2(3 \times 139^2 + 2 \times 139 \times 46 + 46^2)$$

$$\times \frac{\pi \times 12^3}{32 \times 139^3 \times 10^2 \times 16\,000} = 0.0249$$

Using $\frac{1}{4}$-in. round rods for the spiral gives, $p = 1.97$ in.

A $2\frac{1}{2}$-in. horizontal spacing, as indicated on Fig. 3, will give a pitch somewhat less than required and, therefore, is on the safe side. Equation (13) is now used to compute the pitch, p, at the quarter-points and mid-span.

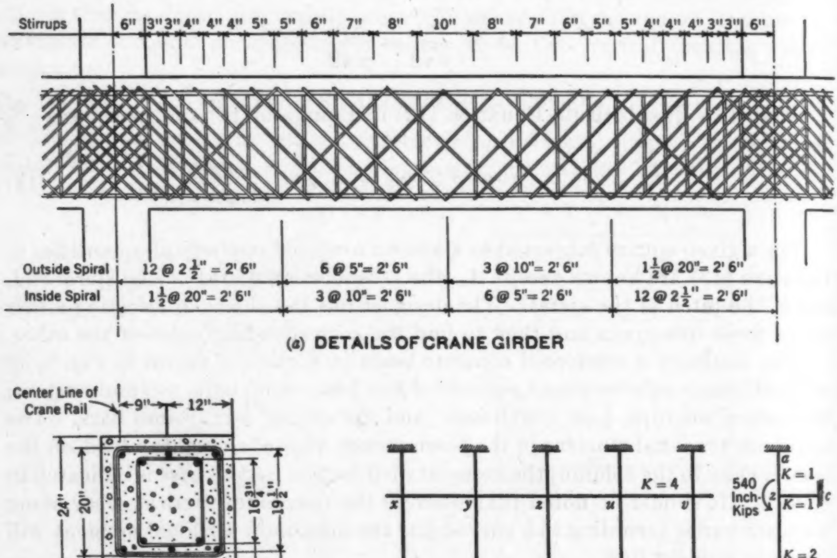


Fig. 3

The Rectangular Section

In the case of a rectangular section, A. A. Eremin, Assoc. M. Am. Soc. C. E., has suggested the use of formulas developed by the writer in 1934[4] with the

[4] *Transactions*, Am. Soc. C. E., Vol. 99 (1934), p. 980.

exception that the value of ρ_e be taken as,

$$\rho_e' = \sqrt[3]{\frac{2\,b^2 d}{\pi\left(3 + \dfrac{2.6}{0.45 + \dfrac{d}{b}}\right)}} \quad \text{..................... (14)}$$

in which ρ_e' = the radius of a circular section equivalent in strength to a rectangular section. To ρ_s (the distance from the center of the section to the

TABLE 2.—DISTRIBUTION OF MOMENTS, FIG. 3

Member	*xa*	*xb*	*xc*	*xy*	*yx*	*ya*	*yb*	*yc*	*yz*	*zy*	*za*
Stiffness	0.08	0.18	0.08	0.33	0.33	0.08	0.18	0.08	0.33	0.33	0.08
Carry-over factor	0.5	0.5	0.5	1	1	0.5	0.5	0.5	1	1	0.5
Fixed-end moment	..	..	..	..	..	..	..	..	..	..	540
First distribution	..	..	..	..	..	..	..	..	..	−180	−40
							←180→				
Second distribution	..	..	..	..	60	14	32	14	60	..	..
		←60→								←120→	
Third distribution	−5	−10	−5	−20	..	..	..	..	..	−40	−9
							←60→				
Fourth distribution	..	..	..	.. 20	20	5	10	5	20	.. 20	..
Total	5	10	5	60	60	19	42	19	−140	−140	−49

Member	*zb*	*zc*	*zu*	*uz*	*ua*	*ub*	*uc*	*uv*	*vu*	*va*	*vb*	*vc*
Stiffness	0.18	0.08	0.33	0.33	0.08	0.18	0.08	0.33	0.33	0.08	0.18	0.08
Carry-over factor	0.5	0.5	1	1	0.5	0.5	0.5	1	1	0.5	0.5	0.5
Fixed-end moment	←540→		..	..	..	..	..	..	..	..	..	..
First distribution	−100	−40	−180	..	..	..	..	..	..	..	..	..
						←180→						
Second distribution	..	..	..	60	14	32	14	60	..	..	..	..
	←120→										←60→	
Third distribution	−22	−9	−40	..	..	..	..	..	−20	−5	−10	−5
						←60→						
Fourth distribution	..	..	.. 20	20	5	10	5	20	.. 20	..	..	..
Total	−122	−49	−400	−140	19	42	19	60	60	−5	−10	−5

spiral reinforcement) should be assigned the value of ρ_e' less the distance from the center of the spiral to the outside of the beam. The efficiency coefficient should also be somewhat reduced; in view of the lack of experimental data for rectangular sections, however, the writer suggests that the value of two-thirds be used also for rectangular beams, and that the same weight of spiral steel be placed on all four sides.

It would appear that in a rectangular section, designed in this manner, the reinforcement would be over-stressed along the center of the long side and under-stressed along the center of the short side. To the writer, however, it seems a reasonable assumption that, to some extent, the presence of equal quantities of spiral steel in both sides will re-distribute the shearing stresses to the advantage of the long side. For rectangular sections nearly square $\left(\frac{d}{b} \lesseqqgtr 1.25\right)$, the writer believes the method outlined to be safe.

Conclusions

The paper outlines the various steps in the design of reinforced concrete in torsion. The Cross method provides a useful tool for determining torsional moments in a space structure.

The square section, which is a good torsion shape, can be reinforced effectively against torsional stresses by the 45° spiral. The quantity of reinforcement can be found by a procedure similar to the accepted practice for designing web reinforcement for vertical shear. This procedure can also be followed for rectangular beams.

APPENDIX

Notation

The following symbols, introduced in the paper, conform essentially with "Symbols for Mechanics, Structural Engineering, and Testing Materials" compiled by a Committee of the American Standards Association, with Society representation, and approved by the Association in 1932[5]:

A = area; A_s = cross-section area of steel bars;
b = breadth, or the short side of a rectangular section;
c = perimeter of a spiral;
d = depth, or the long side of a rectangular section;
E = modulus of elasticity; E_s = modulus of elasticity in shear;
F = a reinforcement efficiency factor that allows for surface tension taken by reinforcement;
I = moment of inertia;
K = stiffness; K_F = flexural stiffness; K_T = torsional stiffness;
L = length of a member;
M = bending moment; an arbitrary moment applied at one end of a member; M_f = fixed-end moment; M_t = torsional moment;
m = a distance from the fixed end of a beam (see, also, n)
N = number of bars, 45° to the axis, cut by a horizontal plane;
n = a distance from the fixed end of a beam (see, also, m);
p = pitch of a spiral;
s = unit stress; s_c = permissible unit tensile stress for concrete; s_s = permissive unit tensile stress for steel;

[5] A. S. A.—*Z10a*—1932.

T = torsion factor = polar moment of inertia in a circular section;
V = total vertical shear;
v = unit shearing stress; v_m = maximum shearing stress; v_m' = maximum unit vertical shearing stress; v_e = shearing stress along the edge of a cylinder, produced by a twisting moment, M;
ρ = radial distance in a horizontal cutting plane, measured from the axis of a cylinder: ρ_e = radius to the edge; ρ_s = radial distance to the steel reinforcement; ρ' = radius of a circular section, equivalent in strength to a rectangular section.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS

ECONOMICS OF THE OHIO RIVER IMPROVEMENT

BY C. L. HALL,[1] M. AM. SOC. C. E.

SYNOPSIS

Brief descriptions of the improved Ohio River are given in this paper, including methods used in navigating it, and the character of the freight borne upon it. The problem presented is then stated, namely, is the public compensated for the heavy national expenditures on the improvement? Commercial navigation costs are determined as accurately as possible, on a ton-mile basis, for each class of freight. Government costs are analyzed on a ton-mile basis, applicable to all classes of freight. The sum of these two costs is compared with rail rates for various commodities, both analytically and graphically. The conclusion is reached that the public has been compensated for its expenditures on the Ohio River improvement, and that this fact tends to become more and more evident every year.

PHYSICAL SITUATION

The Ohio River is formed by the confluence of the Allegheny and Monongahela Rivers, at Pittsburgh, Pa., and discharges into the Mississippi River at Cairo, Ill. The distance from Pittsburgh to the mouth, measured by water, is 981 miles. (The mile used in this paper is the statute mile of 5 280 ft, or 1 609.4 m. The ton is the short ton of 2 000 lb, or 907.18 kg.) The river is improved by four fixed dams, and forty-two movable (or navigable) dams. Two of the fixed dams, recently constructed, have movable crests.[2] At fixed dams, traffic always moves through the locks; at movable dams, it moves through the locks at low water. At medium and high stages, the movable dams are lowered, and traffic moves in the open river. The percentage of time during which open-river traffic is possible, varies from 28 at the up-stream movable dam, to 50 at the down-stream dam. At the Falls of the Ohio, at Louisville, Ky., there is a very high movable dam; and traffic moves through the locks there for 95%

NOTE.—Discussion on this paper will be closed in **February, 1938,** *Proceedings.*

[1] Col., Corps of Engrs., U. S. A.; Commanding 1st Engrs., U. S. A., Fort DuPont, Delaware.

[2] Descriptions of the project are contained in the Annual Repts., Chf. of Engrs., U. S. Army; in Rept. for 1935, pp. 1043–1047.

of the time. Fixed dams and the dam at Louisville are provided with twin locks, whereas the other movable dams have a single lock. The standard lock dimensions are 600 ft by 110 ft. The operation of the system has been described by the writer elsewhere.[3] The minimum depth of the channel is 9 ft, allowing a loaded draft of a little more than 8 ft. However, except at times of extreme low water, barges can be loaded to 9 ft.

Traffic is subject to interruption by high water, ice, and fog. Interruptions by high water at fixed dams are due to the passage of the water over the lock-gates, preventing their functioning. Technically, there are no interruptions by high water at movable dams, but, at very high stages, navigation becomes dangerous. Moreover, the economic functioning of terminals becomes impossible when the river is out of its banks. Ice in sufficient quantities makes navigation impossible. Ice at low water may also compel the lowering of movable dams, making the useful depth too small for economical navigation.[4] At certain seasons of the year, fog at night and in the early morning causes a suspension of traffic.

Conditions from year to year differ to such an extent that it cannot be said that there is any normal number of days during which traffic is interrupted. The median number of days, at Cincinnati, Ohio, about half-way down the river, for the years 1919 to 1935, inclusive, was as follows: By high water, less than one day; by ice, three days; and by low water following ice, less than one day. (These data were obtained from study of official records of the U. S. Engineer Office, at Cincinnati.) Traffic is tied up on account of fog about 80 hr per yr, but never for more than 12 hr at a time. (Little has been published on fog. The information used has been secured by the writer in private discussion with operators.)

The economics of Ohio River transportation are materially influenced by the improvement of five principal tributaries: The Allegheny and Monongahela Rivers, which form the Ohio, at Pittsburgh; the Kanawha River, which joins the Ohio, at Point Pleasant, W. Va. (Mile 265.7); the Cumberland River, at Smithland, Ky. (Mile 920.4); and the Tennessee River at Paducah, Ky. (Mile 934.5). All these tributaries have been at least partly improved by the construction of locks and dams, thereby permitting year-round navigation from and to the Ohio River. Of lesser importance are the other improved tributaries: Muskingum, Little Kanawha, Big Sandy, Kentucky, and Green Rivers.

History of the Improvement

The first appropriation for the improvement of the Ohio River was made by Congress in 1824. Allotments to the year 1874 were for an open-channel improvement, consisting of the construction of dikes, removal of snags and wrecks, and some dredging. This work permitted navigation at medium and high stages. During this period, the Government also purchased an interest in the Louisville and Portland Canal, which passes traffic around the Falls at Louisville. In 1874, a plan was proposed for canalizing the Ohio River by

[3] Rept. No. 12, 1st Section, XVIth International Cong. of Navigation, Brussels, Belgium, 1935.

[4] Rept. No. 12, 1st Section, XVIth International Cong. of Navigation, Brussels, Belgium, 1935, p. 13; also, "Pack Ice and Movable Dams," by C. L. Hall, M. Am. Soc. C. E., *Military Engineer*, July–August, 1934, Vol. XXVI, No. 148, pp. 245–247.

means of a system of movable dams, in order to secure a depth of 6 ft. The first dam constructed under this plan was completed in 1884, at Davis Island, about 5 miles down stream from Pittsburgh. Work under the plan of 1874 progressed slowly. From time to time, the plan was modified. In 1910, a project was adopted by Congress, which provided for the canalization of the river by a system of movable dams, which established a channel depth of 9 ft, substantially as subsequently executed. This project was completed[5] in 1929. Except for some early work on the Louisville and Portland Canal, the entire improvement has been in charge of the Corps of Engineers, United States Army.

Nature of Traffic

Fig 1. shows, graphically, the traffic on the Ohio River for the calendar year 1934.[6] Although the tonnage differs from year to year, the general picture presented by Fig. 1 remains fairly constant, the principal alterations in recent years being the considerable increase in movement of petroleum products, and a decrease in the short-haul sand and gravel movements. The character of Ohio River commerce can probably be best explained by a discussion and amplification of the information contained in Fig. 1.

Steel is the most valuable cargo carried on the river, and the second most important in ton-mileage (26%). Manufactured steel is habitually loaded in barges at points in the Pittsburgh District. A small portion of it is discharged at Ohio River ports, but most of it is consigned to Lower Mississippi River towns. Practically all this steel is moved by contract carriers, or by private carriers owned by the steel companies, and is down-stream movement. There is a return cargo in the form of scrap steel, gasoline, sulfur, and fluorspar, which is collected at Lower Mississippi and Lower Ohio ports, for consignment to Upper Ohio River ports.

Coal forms the largest item, both in tonnage and in ton-mileage (42%), in Ohio River traffic. About 4 600 000[7] tons of coal pass out of the Monongahela every year. This coal is practically all unloaded and consumed at manufacturing plants in the Pittsburgh District. Almost none of it moves as far down stream as Wheeling (Mile 90.5). There is a movement from tipples in the pools of Locks Nos. 13 and 14 (Miles 96.1 to 114.0) in both directions, but principally up stream, to near-by manufacturing plants. In the stretch of river from below Wheeling to the mouth of the Kanawha River (Mile 265.7), practically no coal moves. A considerable tonnage of coal moves out of that river by water to Middle Ohio River plants. A much larger quantity moves from Huntington, W. Va., tipples (Mile 308.3). Some of this is discharged at plants in the Ironton-Portsmouth District (Miles 327.2 to 356.0). Much the greater proportion is unloaded at Cincinnati (Mile 470.2), and just below. A small quantity reaches Louisville (Mile 603.7). From Louisville to Caseyville, Ky. (Mile 871.2), a small quantity of coal moves on the river, principally from the Green River (Mile 784.2). From Caseyville to the mouth, there is a

[5] Annual Rept. of the Chf. of Engrs., U. S. Army, 1935, pp. 1043–1044; House Doc. No. 306, 74th Cong., 1st Session, pp. 134–137.

[6] From an official chart prepared from information submitted by carriers in accordance with law.

[7] Obtained by averaging the 1930–1934 tonnages of the Monongahela River, Annual Repts., Chf. of Engrs., U. S. Army, fiscal years, 1931–35, Pt. 2.

relatively small movement of coal consigned to Lower Ohio and Mississippi River towns. Practically all the coal moved on the river is transported by private carriers; those in the Pittsburgh District are generally owned by steel companies, and those in other parts of the river by coal companies. Except for the small tonnage which moves up stream from the pools of Locks Nos. 13 and 14, and on the Lower Ohio River, all coal moves down stream. There are no appreciable return cargoes on coal barges.

Petroleum products (11% in ton-mileage) move in both directions in special tank barges, normally from riparian storage tanks to riparian delivery tanks. However, the major movement is in the Lower Ohio River, from storage tanks along the Mississippi River and its other tributaries. These products are usually transported in private barges, but many of the barges are propelled by towboats engaged in contract or common carriage. About 75% of the movement (in ton-mileage) is up stream.

Sand and gravel (6% in ton-mileage) is dredged from sand and gravel bars in the river and moves in both directions in privately owned barges for comparatively short distances to delivery points.

Miscellaneous freight (13% in ton-mileage) moves in packet-boats, and also in barges owned by common and contract carriers, although some fluor-

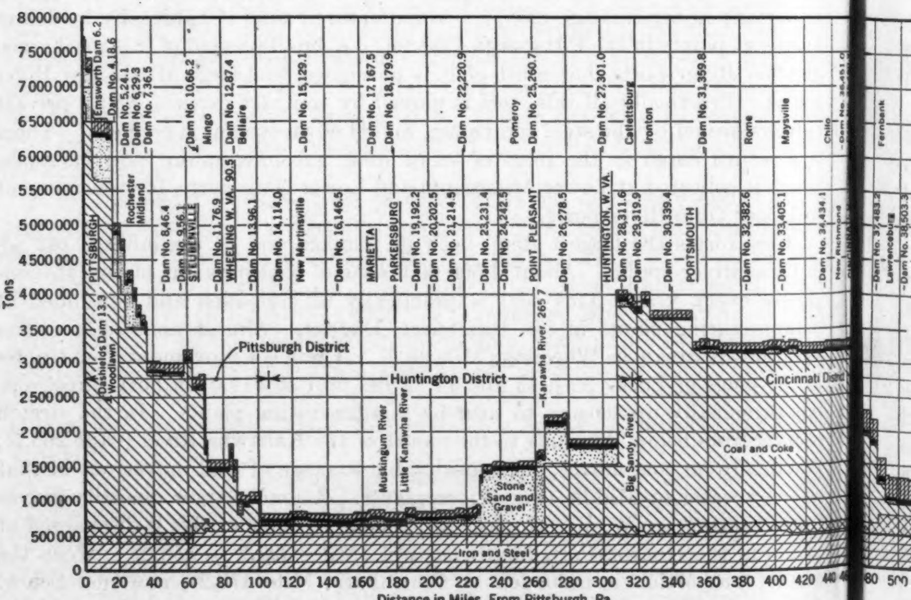


FIG. 1.—OHIO RIV[...]ONNAGE

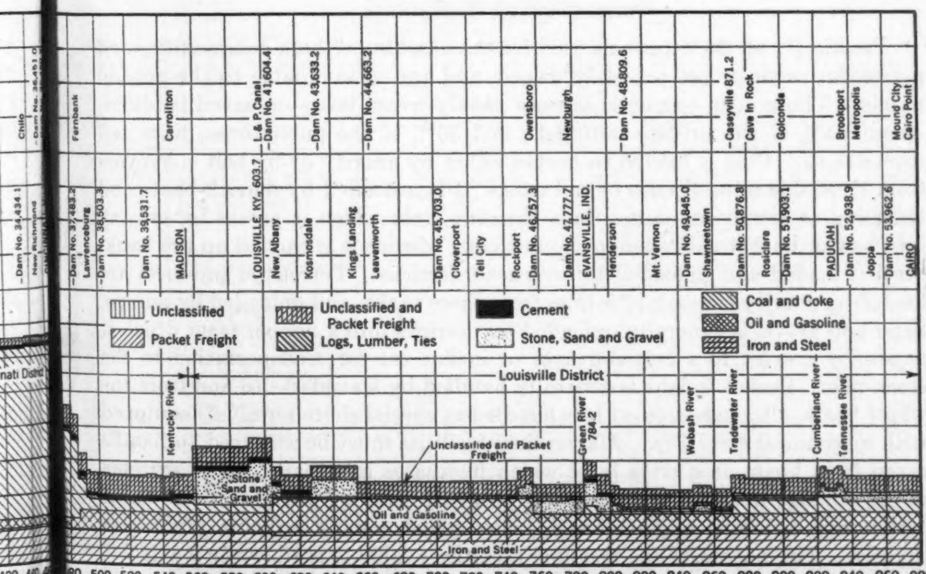
spar from Lower Ohio River ports is carried up stream by private carriers owned by steel companies. Miscellaneous freight moves in both directions.

Less than 3% of the traffic (in tonnage) on the Ohio River moves by common or quasi-common carrier. About one-half of this comparatively small tonnage moves from one port to another on the river, on unpublished rates,

or on rates frequently changed on short notice, and the remainder, the river-rail common-carrier traffic, moves on rates "established" in the railroad sense of the word.

Character of Boats

Bulk freight is habitually carried in steel barges normally about 26 ft by 175 ft, or in wooden barges about 26 ft by 135 ft. These carriers are propelled in fleets of from six to eighteen barges by stern-wheel or screw towboats, which invariably push rather than tow the barges. In the transportation of coal in wooden barges, tows of twenty-four barges are not infrequent. The towboats vary from 30 hp to 2 200 hp, with an average draft of from 4 to 6 ft. Package and perishable freight may be carried either in tows of closed barges (a method which is increasing in popularity among river operators), or on packet-boats, similar in type to the packets of the Nineteenth Century. Petroleum and other liquid products are carried in specially constructed tank barges. A few of the towboats are of the tunnel screw type. This type of towboat is becoming increasingly popular on Western rivers as is the Diesel engine instead of the steam engine. Many of the newer vessels on the Ohio River are of the Diesel, screw type.[8]



OHIO RIV ONNAGE GRAPH, 1934

The size of the locks at Ohio River dams has been a controlling factor in the construction of barges. A tow of barges, 26 ft wide, four abreast (which is standard on the Ohio River), occupies a minimum of 104 ft in a lock 110 ft

[8] The technique of inland navigation is described in the reports of the Towboat Board: H. R. Doc. No. 857, 63d Cong., 2d Session; H. R. Doc. No. 108, 67th Cong., 1st Sess.; and mimeographed report of June 17, 1929. Information on the character of vessels now using the Ohio River has been obtained from the U. S. Engineer Office, at Cincinnati, Ohio.

wide. In the purchase of new barge equipment, there is a trend toward the construction of barges 35 ft wide for assembly in tows of three abreast. The nature of the floating equipment using the river is shown in Table 1.[9]

TABLE 1.—Trips and Drafts of Vessels, Ohio River, 1935

Draft, in feet	Up-Bound Travel				Down-Bound Travel			
	Steamers	Motor vessels	Barges	Total	Steamers	Motor vessels	Barges	Total
9	41	....	3 273	3 314	20	....	6 922	6 942
8	140	2	2 359	2 501	132	1	6 509	6 642
7	1 156	219	2 810	4 185	1 168	72	5 210	6 450
6	2 356	902	4 698	7 956	2 216	771	5 764	8 751
5	3 074	970	1 896	5 940	2 649	942	1 937	5 528
4	2 045	1 651	1 205	4 901	1 728	1 666	843	4 237
3	207	3 246	7 910	11 363	185	3 172	589	3 946
2	1	640	11 975	12 616	1	602	5 237	5 840
1	....	....	5 022	5 022	....	....	1 933	1 933
Total	9 020	7 630	41 148	57 798	8 099	7 226	34 944	50 269
Total net registered tonnage	2 095 863	579 533	25 713 164	28 388 560	1 780 229	347 583	21 339 623	23 467 435

Character of Terminals

Practically all the terminals used for the handling of bulk commodities and petroleum products are privately owned, and are not available to the general public. Those open to public use are mostly wharf boats or paved landings. About 65% of the private terminals, and 30% of the public ones, have rail connections. Coal is loaded to barges either by gravity or by belt conveyors from river-side mine tipples or rail cars. It is unloaded by derrick-boats and traveling or revolving cranes, into hoppers, from which it passes by gravity into belt or bucket conveyors or incline cars, whence it is carried up the bank. Steel is loaded and unloaded by overhead derricks. Petroleum products are loaded by gravity through pipe lines from shore tanks, and unloaded by pumps. Sand and gravel is generally unloaded by derricks into a hopper from which it passes by gravity to a belt conveyor or incline car for transportation to the stock-pile. Packet freight is generally handled by stevedores to and from the wharf boats, although some of it is handled at special shore terminals equipped with mechanical elevators. All terminal facilities must be arranged to handle cargo from boats at a river level which fluctuates irregularly. The extreme range of stage during which traffic is normally handled varies from 16 ft, at Pittsburgh, to 44 ft, at Cincinnati, below which point the figure diminishes slightly. The nature of the terminal facilities available is shown in Table 2.[10]

The Problem

Freight moves on the Ohio River because it can be transported more cheaply by water carriers than by land carriers. It can be transported at low rates because engineering works have been built on the river at the expense of

[9] From Rept. of Corps of Engrs., U. S. Army, Ohio Rivers, 1935.

[10] "Transportation in the Mississippi and Ohio Valleys," Transportation Series Nos. 2, Repts., Corps of Engrs., U. S. Army, 1929, p. 123.

the tax-payer. Since no tolls are charged, the Treasury derives no direct return. The public obtains an indirect return in the reduced net cost of water-transported articles consumed. The problem is to compare the engineering costs of the improvement with the value of the public benefits. This Ohio River question, of course, is a part of a larger national inland waterways problem,[11] which is now arousing great interest among American engineers and economists. (For example, John S. Worley, M. Am. Soc. C. E., in 1936,[12] stated that "the operation on this [Ohio] river gives little encouragement that any justification will be found.") It is also receiving international attention, the International Congress of Navigation in 1935 having particularly requested its members to prepare reports on the economic value of improved inland waterways.[13]

TABLE 2.—Terminal Facilities (Except for Motor-Boats) on the Ohio River

Number	Type of freight	Shelter	Mechanical appliances	Rail Connections	
				With	Without
		(a) Open to the Public			
6	General	Paved wharf or landing	....	6	0
4	General	Warehouse	Crane	4	0
3	Coal, steel, and cement	....	Electric whirler or crane	3	0
1	General	Shed	....	..	..
12	General	Wharf boat	....	..	..
1	General	Movable warehouse and wharf building	....	..	..
1	General	Wharf boat and warehouse	....	..	..
10	General	Paved wharf or landing	....	..	..
2	Miscellaneous	Wharf boat	....	..	..
1	Miscellaneous	Rolling transit shed	....	..	..
41	Totals	....	....	..	..
		(b) Not Available to the General Public			
54	Petroleum products	....	Pipe line or pumps	27	27
78	Coal and coke	....	Clam-conveyor or tipple	42	36
8	Iron and steel	....	Crane	8	..
5	Cement	....	Pump, conveyor, crane, or hand trucks	3	2
57	Sand and gravel	....	Clam or conveyor	44	13
10	Coal, sand, and gravel	....	Clam or incline	8	2
4	Coal and steel	....	Crane or incline	4	..
1	Brick	....	Conveyor	1	..
24	General and bulk freight	....	Crane, incline, or slings	16	8
12	Building and repair of boats	....	Marine ways or floating docks	4	8
11	Logs and lumber	....	Crane or incline	10	1
5	Miscellaneous building materials	....	Crane, clam, or conveyor	4	1
6	Grain and feed	....	Tipple or conveyor	6	..
275	Totals	....	....	177	98

[11] "The High Cost of Inland Water Transportation," by S. L. Wonson, M. Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., September, 1937, pp. 1246–1258.

[12] "Regulating Transport," by John S. Worley, *Engineering News-Record*, July 9, 1936, p. 52.

[13] Resolutions of the XVIth International Cong. of Navigation, *Bulletin*, Permanent International Assoc. of Navigation Congresses, No. 20, p. 53.

Periods of Time Covered

This paper, so far as its facts, as distinct from their analysis, are concerned, is a sequel to a report of 1926, by C. W. Kutz, M. Am. Soc. C. E., which was presented later to the public under the title of "The Relations of the Ohio River and Its Tributaries to Transportation in the United States."[14] No attempt has been made to restudy the investigations covering the period from 1905 to 1925 treated by General Kutz. However, improved collection of commercial statistics by the Engineer Department now permits the acquisition of information in regard to ton-mileage, as distinct from tonnage. When no such information was available, the segregation of credit for commerce moving on two or more rivers had to be made separately for each form of haul. Now, credit for commerce not moving on a single river can easily be determined from ton-mileage charts, after an average cost per ton-mile for the commodity has been established.

The commercial statistics are by calendar years, whereas Federal expenditures are reported by fiscal year (July 1 to June 30). This six-months time lag of benefits over costs will obviously be of no importance over a long period, and has been disregarded. Commercial statistics are closed as of December 31, 1934, and expenditures as of June 30, 1934, since it was convenient to establish limiting dates based on publications available at the time the study was initiated. (See heading, "Trends of Commerce" for a discussion of the most recent data.)

Assumptions

As is necessary in all generalizations from data taken from sources of diversified origin, certain assumptions have been made in the statistical work of this study:

> Assumption (1).—The entire reduction of costs in transportation of freight, due to the existence of an improved waterway on the Ohio River, is passed on to the public in the form of lower prices of the goods transported, and hence becomes a public saving.

Assumption (1) complies with the principles of Nineteenth Century economics. When goods are manufactured at *A* and sold at *B*, the savings in transportation over the line, *A B*, are reflected in the lower price of goods at *B*. The assertion is undoubtedly true in a perfectly free economy. It may be doubted whether a perfectly free economy ever has existed, but it certainly does not exist now on the Ohio River. The two most important items of commerce—coal and steel—are manufactured, transported, and, to a large extent, distributed by firms which, on a small scale, correspond to what the Germans call "vertical trusts." Thus, the same firm, under different forms of capitalization, will own coal mines, steel plants, and steel yards, which are essentially jobbers' supply stations. They will also own towing companies for the transportation of coal and manufactured steel. Except over the comparatively short distances during which their raw materials move by common carrier, the manufacture, distribution, and sale of water-borne steel

[14] *Transactions*, Am. Soc. C. E., Vol. 89 (1926), pp. 1106–1122.

is handled under one corporate management. The same applies to coal companies, which own mines, tipples, transportation lines, and storage yards.

Free competition between the point of production and the point of transfer to the retailer does not exist. At the same time, there is great competition between corporate enterprises having more or less widely separated points of production, and almost adjacent points of distribution. This tends to set up free economic competition between the corporate enterprises, under which the economies due to water transportation tend to be passed on to the consumer. However, the extent to which this saving is transferred is not susceptible of accurate determination. Under certain circumstances it may become zero. The following testimony[15] of a coal operator is apposite:

> "The cost of the coal to our customers at the inland destinations is identically the same, regardless of whether the coal is shipped all-rail from the mines or ex-river from Cincinnati or Addyston."

The comparatively large industrial consumers, however, are undoubtedly able to secure reductions in cost of the delivered coal by bargaining. Where these consumers are utilities, and, to some extent, where they are not, this saving is passed on. Indeed, it would be surprising in a competitive market if savings in cost of transportation were not reflected in reduced retail costs. In course of time, as the number of firms technically fitted to engage in water transportation becomes greater, the assumption will become more and more true. It will thus eventually become accurate. As far as Assumption (1) is in error, the error is in favor of the waterway.

Assumption (2).—The reduction of cost in transportation of freight is equal to the rail cost of transporting the freight actually carried by water, minus the water cost of transporting that same freight.

In general, water-borne freight is some form of bulk commodity, habitually transported by rail where no water route is available. It is logical, therefore, to compare rail and water costs. The difficulty is that rail costs cannot be determined, and it is necessary to use rail rates. If one could use water rates, comparison would still be logical. However, except for the very limited common carrier traffic, water rates are as difficult of determination as rail costs. Therefore, it has been necessary to compare rail rates (in the general case), with water costs, adding to the latter a 6% return on the invested capital. The water costs consist of the line-haul cost of freight, plus the rail rate from the origin to one port and from the other port to the destination (if there has been any rail travel), together with a carefully determined terminal differential. Where the terminal differential is in favor of the waterway, as is apparently the case with liquid petroleum products, none has been included. Assumption (2) is reasonably accurate.

Assumption (3).—The reduction in the cost of ferriage over a waterway, caused by its improvement, is not a public saving.

Ferry transportation is obviously cheaper in a canalized river than in an open one. On the other hand, owing to the increased number of bridges, this

[15] Testimony of H. E. Webster for the West Virginia Coal & Coke Corporation, Interstate Commerce Comm. Docket No. 25 933.

form of traffic is of diminishing importance. There seems to have been no reduction in tolls as a consequence of canalization, so that no direct benefit has been passed on to the public. If there is any error in Assumption (3) it is against the waterway.

Assumption (4).—The United States Engineer Department costs include the entire Federal cost of the improvement.

Certain expenditures of the Federal Lighthouse Service and of the Steamboat Inspection Service are obviously for the benefit of Ohio River transportation. It has proved impossible in practice to segregate these costs for each waterway, because the books of the Services do not show any such breakdown. The costs are so trifling when compared with those of the Engineer Department that it is not worth while to spend too much time on their study. Assumption (4) is favorable to the waterway.

Assumption (5).—In comparing expenditures and benefits, no allowance has been made for taxes.

There is considerable claim that annual water costs should be charged with an arbitrary percentage of the existing value of the Federal improvement made in the stream. There would be something to say for this if it were applied generally to all carriers moving on public highways. Nobody, however, charges the cost of lighting an airway against an airline. Moreover, there is a countervailing tax benefit, of which no account is taken, the omission of which might be even more adverse to the waterway than an arbitrary tax allowance is beneficial. The carriers pay taxes on terminal properties which would have practically no value if the waterway were not improved. They pay taxes on floating plant which would not exist if there were no waterway. They pay income taxes. The first two forms of tax are presumably reflected in costs, but the income taxes are not. No determination can be made of them, because it would be necessary to set up a counter claim for the reduced income tax of competing forms of transportation. This would create a problem impossible of solution. It is not altogether unfair to state that the attempt to determine the taxable value of the waterway is about as difficult as that of determining the taxable value of Central Park in New York City. The entire value of the real estate abutting on that Park is based on its existence. A large part of the value of all real estate in the Ohio River basin is similarly based on the existence of the improvements. Under such circumstances, there is no rational method of assessing a charge for unpaid taxes against the waterway. There is no method of determining whether or not Assumption (5) is beneficial to the waterway.

To summarize, Assumptions (1) to (5), taken as a whole, may be regarded as favorable to the waterway, but the extent of this prejudice tends to diminish with time and, for all practical purposes, may eventually disappear.

Carrier Costs of Water-Borne Freight

The most difficult problem statistically in any study of water transportation is the determination of the actual costs incurred by the private or contract carrier, in moving bulk products concerning which he is not required to quote

any rates to any one. By an analysis of figures furnished confidentially by various carriers in 1926, General Kutz prepared a graph of the costs of transporting coal by water.[16] Since that date, the movement of coal from the Kanawha River and from Huntington to Cincinnati, has become the greatest single factor in tonnage of any Ohio River movement, except the short haul immediately down stream from Pittsburgh. The private coal carriers in this area decline to furnish cost figures, even confidentially. It is not believed that the Pittsburgh coal carriers have been able to reduce their costs much below those shown in the Kutz study—that is, about 3 mills per ton-mile. The line costs of the Huntington-Cincinnati traffic seem to have been brought to a figure below this point. The best indications available to the writer are that the cost to the most economical carrier is now less than 2 mills per ton-mile. A hypothetical study by a prospective operator, which was shown to the writer, gives a still lower figure.

The steel carriers, although unwilling to be quoted individually, make no particular secret of the fact that the line cost of transporting steel for long hauls on the Ohio River is not more than 3 mills per ton-mile. A large contract carrier has submitted to the writer cost data which indicate that his operating cost for 1934, including a 5% return on the investment, was slightly less than 2 mills per ton-mile. A recent official report[17] cites 3 mills per ton-mile for Ohio River traffic in general, but this includes an arbitrary 20% for terminal costs. In computing the savings of the two all-important items of coal and steel, the writer has, in general, used line-haul costs of 2 and 3 mills per ton-mile for principal coal movements, and about 3 mills per ton-mile for steel. These figures are thus slightly higher than what might be termed the general opinion of the industry. The errors, if any, being in favor of decreased costs, are against the waterway. Although no statistical certainty can be reached, it seems probable that the errors against the waterway caused by assuming too high line-haul costs more than balance the errors in its favor caused by the five assumptions previously stated. The cost of hauling petroleum products is even more difficult to determine. The capital investment in the specialized barges (generally used) is so large a part of the total cost that density of traffic, or factor of use, really settles the matter of profit or loss. The operating costs, of course, are increased by the much greater proportion of up-stream traffic encountered in this trade. The writer has used values for savings in this connection which he regards as conservative, and which were based on quotations received from contract carriers. It is regretted that the confidential character of the basic figures obtained prevents a direct statement of the ton-mile cost upon which the savings given in Tables 3 and 4 were based.

The method of computing sand and gravel savings is sufficiently explained in the Appendix. The terminal costs of handling these products are so large, and the mileage of any particular lot is usually so small, that the line cost of transportation (independent of terminal charges) is not a controlling factor in determining the savings.

[16] *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1118.
[17] Not published.

TABLE 3.—ECONOMIC ANALYSIS OF OHIO RIVER IMPROVEMENT, 1905 TO 1925, INCLUSIVE

Year	COAL															SAND AND GRAVEL			STONE		
	Monongahela River to the Lower Ohio River			Monongahela River to the Upper Ohio River			Kanawha River			Huntington, W. Va.			Other Coal								
	Commerce, in thousands of tons	Savings, in cents per ton	Total savings, in thousands of dollars	Commerce, in thousands of tons	Savings, in mills per ton-mile	Total savings, in thousands of dollars	Commerce, in thousands of tons	Savings, in cents per ton	Total savings, in thousands of dollars	Commerce, in thousands of tons	Savings, in cents per ton	Total savings, in thousands of dollars	Commerce, in thousands of tons	Savings, in cents per ton	Total savings, in thousands of dollars	Commerce, in thousands of tons	Savings, in cents per ton	Total savings, in thousands of dollars	Commerce, in thousands of tons	Savings, in cents per ton	Total savings, in thousands of dollars
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)
1905	3 294	60	1 988	....	..	..	1 461	2	32	..	..	..	599	20	120	2 500	10	..	100	15	15
1906	3 225	60	1 919	....	..	..	1 176	3	35	61	20	12	409	20	82	800	10	80	100	15	15
1907	3 344	60	2 005	....	..	..	1 667	5	77	240	20	48	318	20	64	1 030	10	103	100	15	15
1908	2 406	63	1 505	....	..	..	965	5	52	326	20	65	300	20	60	950	10	95	100	15	15
1909	1 914	61	1 169	....	..	..	1 065	6	70	265	20	53	855	20	171	1 050	10	105	100	15	15
1910	1 657	60	996	117	6.4	13	1 248	7	89	386	20	77	357	20	71	980	10	98	546	15	82
1911	2 574	65	1 666	243	5.4	22	1 346	8	108	442	20	88	1 864	20	372	2 328	10	233	218	15	33
1912	1 731	64	1 110	262	5.8	26	1 276	9	112	339	20	68	1 301	20	260	1 184	10	118	24	15	3
1913	1 663	64	1 076	957	5.4	93	1 343	10	132	227	20	45	2 317	20	463	807	10	81	25	15	4
1914	1 099	64	709	804	5.5	80	1 053	10	102	336	20	67	937	20	187	1 831	10	183	193	15	29
1915	1 130	66	747	1 393	5.4	150	1 205	12	140	378	20	76	440	20	88	1 891	10	189	133	15	20
1916	653	68	448	1 237	4.0	99	1 381	12	172	643	20	129	....	..	..	2 011	11	220	30	17	5
1917	330	82	271	1 228	4.4	108	1 429	13	181	222	20	45	....	..	..	1 202	12	144	9	19	2
1918	....	..	....	1 883	6.2	233	1 092	24	264	36	20	7	795	20	159	1 688	13	219	20	21	4
1919	....	..	....	1 280	8.3	213	750	17	130	85	20	17	1 351	20	270	1 038	14	145	19	23	4
1920	....	..	....	1 735	8.3	318	1 323	10	132	528	60	317	3 064	20	613	1 217	15	182	23	25	6
1921	....	..	....	787	13.7	216	939	48	458	433	60	260	2 274	20	455	1 624	16	260	163	25	41
1922	....	..	....	1 292	13.2	393	712	37	266	234	40	93	1 954	20	390	896	17	152	10	25	2
1923	....	..	....	2 794	13.4	860	1 293	41	535	422	40	169	1 533	20	307	1 202	18	216	171	25	43
1924	....	..	....	2 490	13.4	900	648	44	287	514	40	205	2 160	20	432	3 747	19	712	261	25	65
1925	....	..	....	3 930	10.6	1 739	808	50	406	1 280	40	512	908	24	215	6 855	20	1 354	551	25	136
Total	25 020	..	15 609	22 432	..	5 463	24 180	..	3 780	7 397	..	2 353	23 736	..	4 779	36 831	..	4 889	2 896	..	554

TABLE 3.—(*Continued*)

Year	Logs, Railroad Ties, and Lumber			Iron and Steel			Oil and Gasoline			Packet Freight			Other Freight			Totals		
	Commerce, in thousands of tons	Savings, in cents per ton	Total savings, in thousands of dollars	Commerce, in thousands of tons	Savings, in cents per ton	Total savings, in thousands of dollars	Commerce, in thousands of tons	Savings, in cents per ton	Total savings, in thousands of dollars	Commerce, in thousands of tons	Savings, in cents per ton	Total savings, in thousands of dollars	Commerce, in thousands of tons	Savings, in cents per ton	Total savings, in thousands of dollars	Commerce, in thousands of tons	Savings, in cents per ton	Total savings, in thousands of dollars
	(22)	(23)	(24)	(25)	(26)	(27)	(28)	(29)	(30)	(31)	(32)	(33)	(34)	(35)	(36)	(37)	(38)	(39)
1905	1 000	25	250	....	....	....	....	....	....	500	25	125	3 319	15	498	12 773	24	3 028
1906	1 000	25	250	....	....	....	....	....	....	500	25	125	3 538	15	531	10 809	27	3 049
1907	1 000	25	250	....	....	....	....	....	....	500	25	125	2 461	15	369	10 660	27	3 056
1908	1 000	25	250	....	....	....	....	....	....	500	25	125	1 490	15	223	8 037	28	2 390
1909	1 000	25	250	....	....	....	....	....	....	500	25	125	1 206	15	181	7 956	25	2 139
1910	1 013	25	253	126	50	63	1	50	0.5	500	25	125	3 831	15	575	10 760	21	2 443
1911	955	25	239	31	50	16	11	50	5.5	500	25	125	1 260	15	189	11 772	25	3 096
1912	271	25	68	10	50	5	24	50	12	500	25	125	76	15	11	6 999	26	1 919
1913	226	25	56	84	50	42	11	50	6	500	25	125	252	15	38	8 413	25	2 160
1914	667	25	167	39	50	19	4	50	2	500	25	125	127	15	19	7 589	21	1 690
1915	175	25	44	21	50	10	4	50	2	568	25	142	9	15	1	7 346	21	1 609
1916	70	25	17	7	60	4	6	60	3	412	30	124	4	18	1	6 455	18	1 222
1917	90	27	24	1	70	1	8	70	6	430	35	150	12	21	2	4 962	20	934
1918	190	29	55	22	80	18	21	80	17	423	40	169	3	24	1	6 171	20	1 146
1919	50	31	16	17	90	16	76	90	68	328	45	147	10	27	3	5 004	21	1 029
1920	393	33	130	355	100	355	46	100	46	664	50	332	34	30	10	9 382	25	2 441
1921	622	33	206	53	100	53	28	100	28	365	50	182	19	30	6	7 308	28	2 165
1922	275	33	91	316	100	316	31	100	30	539	50	270	33	30	10	6 292	33	2 014
1923	139	33	46	377	100	377	50	100	50	277	50	138	23	30	7	8 281	35	2 748
1924	104	33	34	550	100	550	82	100	82	220	50	110	91	30	27	10 867	33	3 405
1925	186	33	62	580	130	697	271	150	418	283	50	142	86	30	26	15 737	36	5 706
Total	10 426	..	2 758	2 589	....	2 542	674	....	776	9 509	..	3 156	17 884	..	2 728	183 573	27	49 389

TABLE 4.—Economic Analysis of the Ohio River Improvement, 1926 to 1934, Inclusive

Year	Coal and Coke								
	Monongahela River to the Ohio River*				From the Kanawha River				From Huntington, W. Va.
	Commerce		Savings		Commerce		Savings		Commerce
	In thousands of tons	In thousands of ton-miles	In mills per ton-mile	Total, in thousands of dollars	In thousands of tons	In thousands of ton-miles	In mills per ton-mile	Total, in thousands of dollars	In thousands of tons
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1926	5 735	172 159	10.6	1 825	765	156 925	2.1	329	1 689
1927	5 579	203 133	10.6	2 153	808	165 674	2.1	348	1 676
1928	5 756	234 829	10.6	2 489	962	197 117	2.1	414	1 753
1929	6 029	205 196	10.6	2 175	860	176 217	2.1	370	2 117
1930	6 445	215 113	10.6	2 280	791	162 115	2.1	340	1 952
1931	5 136	179 054	10.6	1 898	855	175 339	2.1	368	1 813
1932	3 915	150 580	10.6	1 596	903	185 098	2.1	389	1 846
1933	5 306	211 446	9.7	2 051	1 055	189 220	2.1	397	2 041
1934	6 106	211 516	9.7	2 052	848	162 435	2.1	341	2 321
Totals	50 007	1 783 026	....	18 519	7 847	1 570 140	....	3 296	17 208

TABLE 4.—(*Continued*)

Year	Coal and Coke (*Continued*)							Sand and Gravel		
	From Huntington, W. Va. (*Continued*)			Other Coal and Coke				Commerce		Savings, in mills per ton-mile
	Commerce	Savings		Commerce		Savings				
	In thousands of ton-miles	In mills per ton-mile	Total, in thousands of dollars	In thousands of tons	In thousands of ton-miles	In mills per ton-mile	In thousands of dollars	In thousands of tons	In thousands of ton-miles	
	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
1926	272 453	2.5	681	524	36 164	3.0	109	8 998	122 336	10.0
1927	282 856	2.8	792	453	31 331	4.0	125	9 187	137 889	10.0
1928	276 425	2.6	719	691	31 751	5.0	159	8 749	125 117	10.0
1929	336 720	4.1	1 380	745	39 279	6.0	236	8 709	125 070	10.0
1930	302 118	3.2	967	581	33 873	7.0	237	8 875	132 322	10.0
1931	282 276	2.8	790	715	33 884	8.0	271	6 526	132 664	10.0
1932	295 113	3.1	915	928	37 460	10.0	375	3 758	87 698	10.0
1933	305 247	3.3	1 007	1 576	58 192	12.0	698	3 355	66 314	10.0
1934	317 436	3.6	1 143	1 529	49 889	12.0	599	3 853	97 439	10.0
Totals	2 670 644	....	8 394	7 742	351 823	....	2 809	62 010	1 026 849	....

* Allegheny River coal included.

On account of the overwhelming importance of the four products, coal, steel, petroleum, and sand and gravel, it has not been deemed necessary to discuss herein the ton-mile cost of transporting the other products upon which the savings have been based. A discussion of each significant item is contained in the Appendix.

TABLE 4.—(*Continued*)

Year	SAND AND GRAVEL (*Continued*)	LOGS AND LUMBER				IRON AND STEEL				OIL AND GASOLINE
		Commerce		Savings		Commerce		Savings		
	Total savings, in thousands of dollars	In thousands of tons	In thousands of ton-miles	In mills per ton-mile	In thousands of dollars	In thousands of tons	In thousands of ton-miles	In mills per ton-mile	In thousands of dollars	Commerce, in thousands of tons
	(20)	(21)	(22)	(23)	(24)	(25)	(26)	(27)	(28)	(29)
1926	1 223	395	9 002	12.0	108	575	238 746	6.5	1 552	318
1927	1 378	474	11 829	11.0	130	1 040	294 389	7.0	2 060	206
1928	1 251	394	11 483	10.0	115	1 413	350 557	7.6	2 664	323
1929	1 250	317	10 363	9.0	93	1 416	405 667	8.1	3 286	590
1930	1 323	336	7 110	8.0	57	1 481	407 780	8.2	3 344	482
1931	1 326	124	2 752	7.0	19	972	425 300	8.4	3 572	607
1932	877	95	2 442	7.0	17	610	289 401	7.0	2 026	964
1933	663	141	3 893	7.0	27	865	329 343	7.4	2 437	1 004
1934	974	167	5 642	7.0	39	844	473 196	8.8	4 164	1 272
Totals	10 265	2 443	64 516	....	605	9 216	3 214 379	....	25 105	5 766

TABLE 4.—(*Continued*)

Year	OIL AND GASOLINE (*Continued*)			STONE				CEMENT			
	Commerce, in thousands of ton-miles	Savings		Commerce		Savings		Commerce		Savings	
		In mills per ton-mile	Total, in thousands of dollars	In thousands of tons	In thousands of ton-miles	In mills per ton-mile	Total, in thousands of dollars	In thousands of tons	In thousands of ton-miles	In mills per ton-mile	Total, in thousands of dollars
	(30)	(31)	(32)	(33)	(34)	(35)	(36)	(37)	(38)	(39)	(40)
1926	35 728	12.5	447	436	12 306	10.0	123	30	2 877	1.9	5
1927	24 238	12.5	303	387	11 088	10.0	111	49	5 363	2.7	14
1928	40 533	12.5	507	426	12 806	10.0	128	80	13 347	5.2	69
1929	74 575	12.5	932	365	12 220	10.0	122	116	14 874	5.7	85
1930	75 289	12.0	903	651	27 191	10.0	272	132	14 341	5.5	79
1931	95 920	12.0	1 151	557	15 005	10.0	150	152	17 698	6.6	117
1932	160 624	11.5	1 847	528	17 380	10.0	174	138	17 352	6.5	113
1933	194 465	10.5	2 042	239	8 333	10.0	83	176	12 416	4.9	61
1934	201 365	10.5	2 114	649	16 189	10.0	162	108	20 728	7.5	155
Totals	902 737	....	10 246	4 238	132 518	....	1 325	981	118 996	....	698

NET SAVINGS

In general, the costs on each of the more commonly used routes for each important commodity were determined by multiplying the ton-mileage moved by the average cost per ton-mile as estimated herein (see heading "Carrier Costs of Water-Borne Freight"), adding the terminal differential between rail and water transfers (except in the case of petroleum where it is in favor of the waterway), and also adding the actual rate for rail haul where (as in the general case) the commodity was hauled either to the river before transportation, from the river after transportation, or both. These costs were deducted from the comparative rail rates, and the gross savings established. From this

TABLE 4.—(*Continued*)

Year	Miscellaneous Freight				Totals			
	Commerce		Savings		Commerce		Savings	
	In thousands of tons (41)	In thousands of ton-miles (42)	In mills per ton-mile (43)	Total, in thousands of dollars (44)	In thousands of tons (45)	In thousands of ton-miles (46)	In mills per ton-mile (47)	Total, in thousands of dollars (48)
1926	289	42 932	3.3	142	19 754	1 101 628	5.9	6 544
1927	269	46 701	3.3	154	20 128	1 214 491	6.2	7 568
1928	388	50 720	3.2	162	20 935	1 344 685	6.5	8 677
1929	690	112 404	3.1	348	21 954	1 512 585	6.8	10 277
1930	611	96 676	3.0	290	22 337	1 473 928	6.8	10 092
1931	613	126 552	2.9	367	18 070	1 486 444	6.7	10 029
1932	632	149 081	2.8	417	14 317	1 392 228	6.3	8 746
1933	992	329 553	2.7	890	16 750	1 708 422	6.1	10 356
1934	939	228 088	2.7	616	18 636	1 783 924	6.9	12 359
Totals	5 423	1 182 707	....	3 386	172 881	13 018 335	6.5	84 648

value, the saving per ton-mile was easily obtained. In a small number of cases where published rates were available, these rates were used in lieu of costs. In general, since it was impracticable to obtain water rates or costs for each year separately, the savings for 1934 were calculated, and the average savings per ton-mile for the years, 1926 to 1934, were apportioned as explained in the Appendix. For the savings prior to 1926 the figures used by General Kutz in an official report were adopted. As explained previously, the break-down by ton-miles was impracticable for earlier years. The Appendix gives details of calculation for each of the more important items.

Official records (Annual Reports of the Chief of Engineers, for each fiscal year, Government Printing Office) give the tonnage and (in recent years) the ton-mileage of each commodity. After the unit saving per ton-mile for each commodity was determined, as shown in Table 3, simple arithmetic gave the commodity savings in dollars. These savings are summarized in Column (39), Table 3, and Column (48), Table 4. The values there given have been used as the net public benefit, with which the annual Federal costs must be compared (Column (15), Table 5).

Government Costs

The determination of the Government costs is the simplest part of this study. Full data are available in the Annual Reports of the Chief of Engineers. The only explanation required refers to a few necessary assumptions in the statistics. The interest charge on Government money has been assumed as 4 per cent. This is high for recent years, but not excessively so. The interest during construction has been assumed at 2% for the year the money was spent; that is, it is assumed that costs were uniform throughout the year. The depreciation has been charged from the year of completion. In every case in which the lock has not been retired from service in less time, the life of the structure has been assumed as 40 yr. A straight-line depreciation has been assumed over that period. Where a lock has either been retired,

or is due to be retired after less than 40 yr in service, the actual life of the lock, in years, has been determined, and a straight-line depreciation taken over that period.

In determining the remaining investment, or the surplus in the cumulated capital account (Columns (23) and (24), Table 5), depreciation of structures (Columns (14), Table 5) has been subtracted. This was done because the annual depreciation of the structures had already been added into the annual charges (Column (15), Table 5), thereby reducing the annual profit by the amount of the depreciation, which, in turn, reduced the net annual capital charges (Column (21), Table 5). Since the depreciation was thus added as a charge for the year, the remaining investment of the Government in the structures was obviously reduced by that amount. With these explanations, Table 5 should become perfectly clear.

Comparison

The graphical analysis in Fig. 2 shows the manner in which at first the annual charges (Column (15), Table 5) increased much more rapidly than the

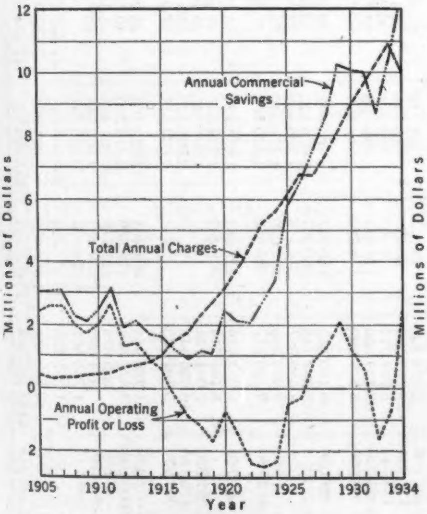


Fig. 2

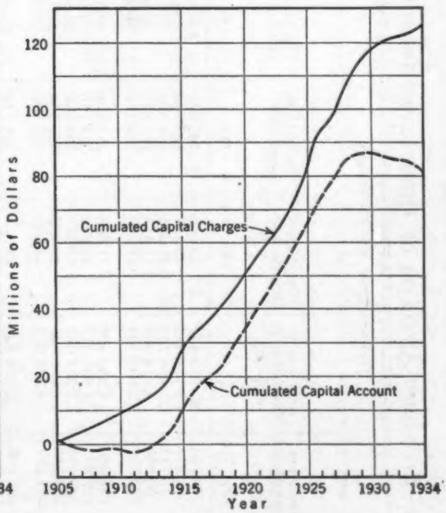


Fig. 3

commercial savings (Column (16), Table 5), then how the savings overtook those charges until the onset of the depression, and then how rapidly these savings have advanced since 1932. Fig. 3 shows the manner in which the capital charges (Column (10), Table 5), as shown on the Government books, have, of course, tended to increase while the capital account (Columns (23) and (24), Table 5), from an investor's standpoint, assuming a public profit from savings, has tended recently to decrease. The relation of savings to expenses is shown analytically in Column (19), Table 5, in which the ratio, 1.0, of savings to expenditures means that the development has barely been

TABLE 5.—ECONOMIC ANALYSIS OF OHIO RIVER IMPROVEMENT; FINANCIAL SUMMARY

Fiscal year	Government Expenditures							Capital Account		
	Lock and dam construction (1)	Open river improvement (2)	Total new works (Columns (1) and (2)) (3)	Open river maintenance (4)	Operating and care (5)	Operating snagboats (6)	Total operation and maintenance (Columns (4), (5) and (6)) (7)	Interest in annual capital expenditures (2% of Column (3)) (8)	Total annual capital charges (Columns (3) and (8)) (9)	Cumulated capital charges (cumulation of Column (9)) (10)
1905	$ 905 738	$ 49 754	$ 955 492	$ 36 806	$ 195 901	$34 688	$ 267 395	$ 19 108	$ 974 600	$ 974 600
1906	1 101 754	72 278	1 174 032	15 437	154 354	33 094	202 885	23 480	1 197 512	2 172 112
1907	1 736 541	47 711	1 784 252	11 354	184 355	35 764	231 473	35 684	1 819 936	3 992 048
1908	1 224 586	90 253	1 314 839	17 415	181 814	37 063	236 292	26 296	1 341 135	5 333 183
1909	1 750 654	283 897	2 034 551	21 760	210 151	36 391	268 302	40 690	2 075 241	7 408 424
1910	1 147 489	133 920	1 281 409	11 508	211 108	34 497	257 113	25 628	1 307 037	8 715 461
1911	1 543 650	147 865	1 691 515	14 299	234 773	31 058	280 130	33 830	1 725 345	10 440 806
1912	1 991 513	290 481	2 281 994	15 279	334 886	31 348	381 513	45 638	2 327 632	12 768 438
1913	2 306 292	382 533	2 688 825	20 202	349 164	25 758	395 124	53 776	2 742 601	15 511 039
1914	4 267 384	261 297	4 528 681	18 545	330 894	36 449	385 888	90 572	4 619 253	20 130 292
1915	7 351 302	427 106	7 778 408	14 051	388 116	51 963	454 130	155 568	7 933 976	28 064 268
1916	4 981 651	177 914	5 159 565	80 408	429 024	36 143	545 575	103 190	5 262 755	33 327 023
1917	3 539 171	185 743	3 724 914	6 803	494 774	50 298	551 875	74 498	3 799 412	37 126 435
1918	3 128 749	25 310	3 154 059	174 917	609 422	44 745	829 084	63 080	3 217 139	40 343 574
1919	4 737 410	66 659	4 804 069	152 285	764 375	55 975	972 635	96 080	4 900 149	45 243 723
1920	5 151 353	29 096	5 180 449	152 247	864 987	32 201	1 049 435	103 608	5 284 057	50 527 780
1921	6 599 146	....	6 599 146	250 512	1 058 112	40 330	1 348 954	131 982	6 731 128	57 258 908
1922	4 459 681	....	4 459 681	276 116	1 100 862	33 049	1 410 027	89 192	4 548 873	61 807 781
1923	5 171 698	51 560	5 223 258	593 070	1 171 445	33 024	1 797 539	104 464	5 327 722	67 135 503
1924	5 747 989	372 788	6 120 777	309 632	1 420 574	57 864	1 788 070	122 414	6 243 191	73 378 694
1925	8 173 393	41 766	8 215 159	427 288	1 617 824	55 375	2 100 487	164 302	8 379 461	81 758 155
1926	8 464 716	....	8 464 716	359 492	1 744 551	47 263	2 151 306	169 294	8 634 010	90 392 165
1927	6 742 825	351 329	7 094 154	....	1 697 547	46 162	1 743 709	141 882	7 236 036	97 628 201
1928	9 618 807	410 135	10 028 942	....	2 096 828	35 121	2 131 949	200 578	10 229 520	107 857 721
1929	5 194 594	606 939	5 801 535	....	2 063 414	57 241	2 120 655	116 030	5 917 563	113 775 284
1930	1 236 476	3 052 438	4 288 914	301 454	2 273 100	45 792	2 620 346	85 778	4 374 692	118 149 976
1931	566 093	2 005 801	2 571 894	278 860	2 818 599	40 621	3 138 080	51 438	2 623 332	120 773 308
1932	979 379	−73 504	905 875	1 011 176	2 994 445	48 420	4 054 041	18 118	923 993	121 697 301
1933	1 224 260	−309 568	914 692	1 959 435	2 803 348	43 705	4 806 488	18 294	932 986	122 630 287
1934	1 474 834	290 727	1 765 561	1 574 972	2 200 894	49	3 775 915	35 311	1 800 872	124 431 159
Sub-total	$112 519 128	$9 472 228	$121 991 356	$8 105 323	$32 999 641	$1 191 451	$42 296 415	....	....	....
Before 1905	9 313 472	4 024 560	13 338 032	1 500 932	2 624 084	402 878	4 527 894	....	....	....
Total	$121 832 600	$13 496 788	$135 329 388	$9 606 255	$35 623 725	$1 594 329	$46 824 309	....	....	....

TABLE 5.—(*Continued*)

Fiscal year	OPERATING ACCOUNT					PROFIT AND LOSS					ALTERATIONS IN CAPITAL ACCOUNT			
							Annual				Net Annual Capital Changes		Cumulated Capital Account	
	Interest charged on investment (4% of Column (23), previous year) (11)	Interest charged on surplus (4% of Column (24), previous year) (12)	Expenditures for operation and maintenance (Column (7)) (13)	Depreciation* (14)	Total annual charges (Column (14) +Column (13) −Column (12) +Column (1)) (15)	Annual commercial savings (Column (39), Table 3, and Column (48), Table 4) (16)	Profit (Column (16) −Column (15)) (17)	Loss (Column (15) −Column (11)) (18)	Ratio of savings to expenditures (Column (16) ÷Column (15)) (19)	Percentage of annual return (Column (12) ÷Column (5), or Column (13) ÷Column (5)) (20)	Increase (Column (9) −Column (17) +Column (18)) (21)	Decrease (Column (17) −Column (9)) (22)	Remaining investment (Column (23) +Column (21) −Column (22) −Column (14) −Column (24))† (23)	Surplus (Column (24) −Column (21) +Column (22) −Column (14) −Column (23))† (24)
1905	$ 124 288	$......	$ 267 395	$ 35 107	$ 426 790	$ 3 027 600	$2 600 810	$......	7.1	266.86	$......	$1 622 210	$ 3 107 254	$......
1906	57 836	......	202 885	35 107	295 828	3 049 300	2 753 472	......	10.3	126.76	......	1 555 960	1 445 937	74 916
1907	......	2 996	231 473	77 575	306 052	3 055 600	2 749 548	......	10.0	68.88	......	929 612	......	926 953
1908	......	37 076	236 292	114 821	314 037	2 389 900	2 075 863	......	7.6	38.92	......	734 728	......	1 546 860
1909	......	61 872	268 302	207 592	414 022	2 138 600	1 724 578	......	5.2	23.28	350 663	......	......	988 605
1910	......	39 544	257 113	207 592	425 161	2 442 700	2 017 539	......	5.8	23.15	......	710 502	......	1 491 515
1911	......	59 660	280 130	230 769	451 239	3 095 700	2 644 461	......	6.9	25.33	......	919 116	......	2 179 862
1912	......	87 192	381 513	362 024	656 345	1 919 200	1 262 855	......	2.9	9.89	1 064 777	......	......	753 061
1913	......	30 120	395 124	414 314	779 318	2 159 700	1 380 382	......	2.8	8.90	1 362 219	......	194 844	......
1914	7 792	......	385 888	414 314	807 994	1 690 300	882 306	......	2.1	4.38	3 736 947	......	3 517 477	......
1915	140 696	......	454 130	470 616	1 065 442	1 608 985	543 543	......	1.5	1.94	7 390 433	......	10 437 294	......
1916	417 488	......	545 575	525 645	1 488 708	1 222 000	......	266 708	0.8	−0.80	5 529 463	......	15 441 112	......
1917	617 664	......	551 875	612 723	1 782 242	933 500	......	848 742	0.5	−2.29	4 648 154	......	19 476 543	......
1918	779 060	......	829 084	725 508	2 333 652	1 145 700	......	1 187 952	0.5	−2.94	4 405 091	......	23 156 126	......
1919	926 244	......	972 635	790 042	2 688 921	1 028 700	......	1 660 221	0.4	−3.67	6 560 370	......	28 926 454	......
1920	1 157 056	......	1 049 435	974 469	3 180 960	2 440 831	......	740 129	0.8	−1.46	6 024 186	......	33 976 171	......
1921	1 359 044	......	1 348 954	974 469	3 682 467	2 165 215	......	1 517 252	0.6	−2.65	8 248 380	......	41 250 082	......
1922	1 650 000	......	1 410 027	1 335 127	4 395 154	2 014 439	......	2 380 715	0.5	−3.85	6 929 588	......	46 844 543	......
1923	1 873 780	......	1 797 539	1 539 941	5 211 260	2 747 756	......	2 463 504	0.5	−3.67	7 791 226	......	53 095 828	......
1924	2 123 832	......	1 788 070	1 629 822	5 541 724	3 405 063	......	2 136 661	0.6	−2.91	8 379 852	......	59 845 858	......
1925	2 393 832	......	2 100 487	1 701 248	6 195 567	5 705 827	......	489 740	0.9	−0.60	8 869 201	......	67 013 811	......
1926	2 680 552	......	2 151 306	1 950 257	6 782 115	6 544 000	......	238 115	1.0	−0.26	8 872 125	......	73 935 679	......
1927	2 957 424	......	1 743 709	2 024 037	6 725 170	7 568 000	842 830	......	1.1	0.86	6 393 206	......	78 304 848	......
1928	3 132 192	......	2 131 949	2 100 464	7 364 605	8 677 000	1 312 395	......	1.2	1.22	8 917 125	......	85 121 509	......
1929	3 404 860	......	2 120 655	2 641 243	8 166 758	10 277 000	2 110 242	......	1.3	1.85	3 807 321	......	86 287 587	......
1930	3 451 500	......	2 620 346	2 876 583	8 948 429	10 092 000	1 143 571	......	1.1	0.97	3 231 121	......	86 642 125	......
1931	3 465 684	......	3 138 080	2 899 950	9 503 714	10 029 000	525 286	......	1.1	0.43	2 098 046	......	85 840 221	......
1932	3 433 608	......	4 054 041	2 901 845	10 389 494	8 746 000	......	1 643 494	0.8	−1.35	2 567 487	......	85 505 863	......
1933	3 420 232	......	4 806 488	2 908 488	11 135 208	10 356 000	......	779 208	0.9	−0.64	1 712 194	......	84 309 569	......
1934	3 372 380	......	3 775 915	2 909 998	10 058 293	12 359 000	2 300 707	......	1.2	1.85	......	499 835	80 899 736	......

* Depreciation computed on the assumption of a 40-yr life for existing locks and dams. Where structures have been replaced, the actual life years were used.

† Columns (23) and (24) for previous year used in formulas for determining figure for Columns (23) and (24).

successful. A higher ratio indicates a profit. In considering these relations between commercial savings and public expenditures, it should not be forgotten that 4% interest has been charged against the entire remaining capital structure for every year. During the depression, railroad companies would

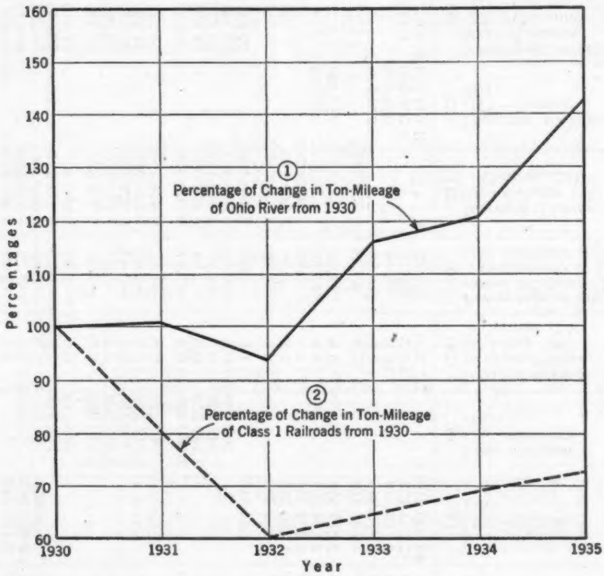


FIG. 4

have regarded themselves as fortunate, if they were able to pay 4% interest on their entire capital. Fig. 4 shows, in percentage form, the relation between the water-borne commerce and railroad commerce, using the commerce of each during the calendar year 1930 as a base. It can be seen that, after

TABLE 6.—RAIL AND WATER COSTS

Item No.	Commodity	COMMERCE		Percentage of total, excluding sand, gravel, stone, logs, and lumber	RAIL COST TO SHIPPER FROM CINCINNATI TO LOUISVILLE			WATER COST TO SHIPPER FROM CINCINNATI TO LOUISVILLE			
		In thousands of tons	In millions of ton-miles		In cents per ton	In mills per ton-mile over water distance	Proportion to sample ton, in mills	Assumed line cost, in mills per ton-mile	Terminal differential, in mills per ton-mile	Over-all cost, in mills per ton-mile	Proportion to sample ton, in mills
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
1	Coal and coke	10 805	741	44.5	149	11.2	5.0	2.3	2.2	4.5	2.0
2	Cement	108	21	1.3	364	27.3	0.4	5.4	6.7	12.1	0.2
3	Iron and steel	844	473	28.4	380	28.5	8.1	6.0	6.7	12.7	3.6
4	Oil and gasoline	1 272	201	12.1	380	28.5	3.4	6.4	0.0	6.4	0.8
5	Unclassified	939	228	13.7	428	32.1	4.4	..	..	20.0	2.7
6	Total	13 968	1 664	100.0	..	..	21.3	..	..	..	9.3

having been checked for a short time in 1932, Ohio River commercial traffic recovered. (It is now (1937) more than 50% greater than it was in 1930.) In 1934 traffic of Class I railroads had not reached a figure equal to 75% of its 1930 figures. (It is now (1937) 12% less than in 1930.)

Table 6 has been prepared for the purpose of comparing the data on public savings by an entirely independent system of calculation. The following items supplement the data in Table 6:

(*a*) Average haul of commodities on Ohio River (1934) in miles 119

(*b*) Water distance, Cincinnati, Ohio, to Louisville, Ky., in miles 133.5

(*c*) Total annual charges, 1934 (Column (15), Table 5) .. $10 058 293

(*d*) Total commerce, Ohio River, 1934 (Column (46), Table 4), in ton-miles 1 783 924 624

(*e*) Government costs, in mills per ton-mile, 1934, $\left(\frac{\text{Item } (c)}{\text{Item } (d)}\right)$ 5.6

(*f*) Government cost plus water cost to shipper (Column (11), Table 6) = total water cost, in mills per ton-mile 14.9

(*g*) Rail cost to shipper, reflected by quoted rates, in mills per ton-mile (Column (6), Table 6) 21.3

(*h*) Saving, in mills per ton-mile, by water, on sample ton 6.4

The average haul was determined from the traffic data of the Ohio River, and two important points on the Ohio River (Louisville and Cincinnati) were selected as being about that distance apart. From the official figures of water-borne traffic, the proportion of each significant commodity in a representative ton was ascertained. Based on the rail rates (in carload lots) for these same commodities, it was possible, in the manner shown in Table 6, to determine the rail rate per ton-mile measured by water for a theoretical ton of representative cargo, moving in a carload lot. By similar methods, the water rate per ton-mile was determined for the same unit. Referring to Column (8), Table 6, the rate for coal and coke (Item No. 1) is that quoted by a water carrier. The rail rate on coal is somewhat higher than the average, but a reduction to 8 mills per ton would affect the final values only slightly. The rate for cement (Item No. 2) is that quoted by a water carrier for a 156-mile haul. Item No. 3 (iron and steel, Column (8)) was used in the 1934 study for the 164-mile haul. A large unit rate is used because a short haul is being assumed. The rate for oil and gasoline is that for an 118-mile haul. No terminal differential (see Column (9), Table 6) is assumed because of the more economical river loading. Item No. 5 in Column (10) is the common carrier average on four major commodities. Terminal handling charges are included in this rate. By dividing the annual expenditures by the total ton-mileage (Item (*e*)), the Government cost per ton-mile was also ascertained. As shown, the rail cost was found to be 21.3 mills per ton-mile, and the total combined water

cost, 14.9 mills per ton-mile, giving an economic public benefit of 6.4 mills per ton-mile for water-borne traffic. Referring to Table 4, it will be seen, from examination of Column (47) for 1934, that the commercial saving by water on the average cargo was determined as 6.9 mills per ton-mile. The economic public benefit by the methods used in Table 4 would be 6.9 mills, less the Government cost of 5.6 mills, or 1.3 mills. Table 6 thus gives a result about five times as favorable to the waterway as Table 4.

Trends of Commerce

For reasons previously explained, commercial costs for the calendar years 1935 and 1936 have not been included in this study. Since its initiation, however, gross figures for commerce for those years have become available. They indicate an increase in tonnage of 9%, and of ton-mileage of 53 per cent. The greater increase in ton-mileage over tonnage is indicative of a trend which has been evident since 1926. In that year the average haul was 55.7 miles; in 1936, it was 108.7 miles. The reasons are obvious. Long-haul traffic (steel, petroleum products, etc.) has increased. Short-haul traffic, especially sand and gravel, has failed to increase proportionately, or has actually decreased. The increase in ton-mileage over 1934, which may be compared with the 9% increase in ton-mileage in the freight moved on Class I railroads for the same years[18] indicates clearly that, when there is any traffic, the Ohio River gets at least its share, and probably more.

At this point it might be well to take a general view of the commercial figures for the ten years since General Kutz presented his paper. They show clearly what has been noticed before, that no one is able to determine in advance the detailed trends of future commerce on a waterway. In that original study, through steel shipments (that is, steel from the Pittsburgh area to Mississippi River ports) were scarcely of enough importance to receive separate treatment. This movement has now (1937) become the most valuable commerce on the Ohio River; and its value seems destined to increase. More and more manufacturers are sending manufactured steel by water; and there is some evidence of an intent to transfer to the inland route steel which now moves from Pittsburgh to the Coast, and thence to the Gulf, coastwise. The movement of water-borne coal follows the general lines already forecast in 1926, except for the unpredicted activity around Wheeling, W. Va., although here, again, it appears that an industrial depression checks the movement of water-borne coal to a markedly smaller extent than it does rail-borne coal. In General Kutz' paper, the only mention of petroleum is in regard to delivery service from a refinery at Parkersburg, W. Va.[19] Petroleum movement has now become a large item in Ohio River traffic, and shows no signs as yet of approaching a saturation point. On the other hand, the common carrier traffic for which great things were hoped, has not developed proportionately. In spite of modern equipment and excellent

[18] Figures were obtained by comparing those given in Annual Reports of the Interstate Commerce Commission.

[19] *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1121.

management, the common-carrier towboat lines find the transportation of privately owned barges at contract rates absolutely essential for their support.

The difficulties of common carriers on Western rivers are in considerable part due to the unbalanced character of the tonnage. On an open channel, the most profitable division of tonnage, from a strictly engineering standpoint, is 8 tons of down-stream traffic to 3 tons of up-stream traffic. For the calendar year 1935 the principal common carrier on the Ohio River informs the writer that his ratio was 21 tons down stream to 20 tons up stream. For the same year the report of the Inland Waterways Corporation indicates that, on the Mississippi River, tonnage was 29% down bound and 71% up bound. On the Upper Mississippi section it reached the extraordinary value of 18% down bound to 82% up bound.[20] From a strictly economic standpoint, the obvious remedy is to increase the up-bound rates and to decrease the down-bound rates until the traffic approaches a proper balance. However, a comparatively slight increase in the up-bound rates might divert traffic from the waterway altogether. Similarly, on rail-water movements, a substantial decrease in the down-stream rates might destroy the water carriers' share of the revenue entirely. Until this subject is adequately studied a remedy for the situation is not apparent. It is evident, however, that a profitable common carrier traffic awaits the discovery of the remedy.

The entire trend of Ohio River commerce is toward an increase—often an enormous increase—in the transportation of low-grade, long-haul, bulk products, in which the consumers' savings are only appreciable after a comparatively long period, and after time has been given for competing producers to engage in water transportation. Only to a very small extent have consumers' benefits become apparent in the immediate reduction in the prices of commodities, as a result of their more economical transportation by carriers entirely disinterested in either the manufacture or distribution of the articles transported. It can be asserted with confidence that the waterway reduces, materially, the proportion of the transportation cost contained in the gross cost of articles carried by water, either in their final state, or as raw materials. It is much more difficult to show, and from all indications it will always remain more difficult to show, that the waterway reduces in the same proportion the price which the consumer pays for the manufactured article.

It is evident at any rate that the pessimistic prophecies of 1926 have not proved true. It was then stated[21] by Frank H. Alfred, M. Am. Soc. C. E., that "it does not seem likely, however, that the completion of the improvement project will result in any considerable increase of river traffic." The data in Table 4 certainly prove the contrary.

The Special Board of Engineer Officers, in its report of December 15, 1906, which constitutes the basic plan for the present improvement of the Ohio River, makes a conservative prophecy, which might well be contrasted with the one by Mr. Alfred. The report states that the commercial saving on a 9-ft project with the then existing commerce would amount to $2 280 000 per year; and that "the ultimate annual saving *** would be several times the

[20] Rept. of Inland Waterways Corporation, 1935, p. 7, Govt. Printing Office, Washington, D. C., 1936.
[21] *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1136.

amount stated."[22] The figures for savings in recent years, Table 4, amount to several times \$2 000 000 per year, and thus confirm the far-seeing estimate of 1906.

NON-STATISTICAL CONSIDERATIONS

Some commentators on transportation refer to the regulating effect of an improved waterway on the rates charged by other carriers. The writer has not been able to discover that the existing rail-rate structure discriminates between points having water competition and points that are without water competition.[23] On the other hand, the general rate structure of the Valley appears to be affected by the availability of the canalized Ohio River. One coal operator, not directly interested in water traffic, has recently made the statement that the cost of coal at Cincinnati would be \$1.00 per ton higher if there was no river. Although this is mere opinion, it seems to coincide with the general opinion. However, the effect on rates is by no means the only effect of the improved waterway on the population of the region. Routes of communication, banking affiliations, the operations of jobbers, and all the other business relations which characterize the true "regions," in the normal sense of the term, have been built up on the assumption of the existence of the improved waterway. If the river were to disappear the population of the water-shed would begin to diminish with great rapidity. Freight rates, like other costs, would rise with the attenuation of the regional market.

Throughout this study no statistical comparison has been made in regard to passenger traffic. Such traffic is only to a very slight extent commercial. It is essentially travel for pleasure. Yet its general importance to the people of the Valley should not be overlooked. During the season of hot weather, boats of all sizes, from canoes to huge excursion boats, constitute a very important element of joy to the people. None of these boats could operate if the river were not improved.

CONCLUSIONS

From both an engineering and an economic standpoint, the canalization of the Ohio River has been successful. This success tends to become greater with time.

APPENDIX

The information in this Appendix supplies confirmatory details for the general information given under the heading "Net Savings."

Coal: Monongahela River to the Ohio River.—The savings per ton-mile for 1934 were 9.7 mills, and for 1925 they were 10.6 mills. As the reduction is believed to have been caused by river-rail movement of coal to Youngstown, Ohio, which was started in 1932, the reduced savings were used for the last two years only.

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[22] H. R. Doc. No. 492, 60th Cong., 1st Session, p. 36.

[23] A similar conclusion was reached by General Kutz, *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1115.

Coal: From the Kanawha River.—In the absence of more accurate information for 1934, savings of 2.1 mills per ton-mile were used throughout.

Coal: From Huntington, W. Va.—The 1934 savings per ton-mile were calculated as 49 cts, and for 1925 as 40 cts. There has obviously been a progressive increase in the saving on this traffic, and the difference in savings was distributed proportionately throughout the years involved.

Other Coal and Coke.—The 1934 savings were 38 cts per ton, and the 1925 savings were 24 cts per ton. It is reasonably clear that this change was due to an alteration in the character of coal movement. In 1934, 69% of the tonnage was from mines in the vicinity of Wheeling, W. Va., which scarcely produced any water-borne coal in 1926. The savings on the Western Kentucky coal, which is the other principal factor in this category, were very much less than those on the Wheeling coal. The savings were interpolated over the years 1926–1934 according to time. The change in the character of coal traffic has been progressive.

Sand and Gravel.—The savings per ton-mile decreased from 14.9 mills in 1925 to 10 mills in 1934. Apparently, the decrease in savings is due to a decrease in tonnage carried per barge employed. No terminal differential was charged on products moved directly to river points for use in repairing adjacent highways, or for the construction of river improvements, because it is more economical to unload a barge at the site of the work than to unload it from railroad cars, and then haul it in trucks from rail-heads. The decreased value of 10 mills per ton-mile is used throughout the period. Any error in this figure is against the waterway.

Logs and Lumber.—The 1925 savings were 12.2 mills per ton-mile. In 1934, they had decreased to 7 mills per ton-mile. The decrease in savings was adjusted proportionately according to time.

Iron and Steel.—The enormous increase in the savings on iron and steel is obviously due to the great increase in length of haul. In other words, the terminal differential chargeable per ton-mile has been greatly reduced. The savings per ton-mile has been proportioned on the basis of ton-mileage.

Oil and Gasoline.—There is no terminal differential on this product, or, if there is, it is in favor of the waterway. As the average haul increased, the savings per ton-mile were not increased. As up-stream traffic becomes a larger portion of the total haul, the savings per ton-mile decreases. There has been a slight decrease in savings per ton-mile since 1926, due to this latter fact. The decrease has been distributed according to the best information available on the change in character of traffic.

Stone.—On this comparatively small shipment, the figure for saving per ton in 1926 was used throughout.

Cement.—This commodity makes its first appearance at this time as a separate item. The savings, however, were estimated in an unpublished document at 1.9 mills per ton-mile in 1925. In 1934, calculations show the savings at 7.5 mills. The increase is due partly to the fact that no terminal differential is charged on cement delivered to riparian construction projects, and partly to a decided increase in volume of traffic. The savings for the years

1925–1934 have been proportioned on the basis of the ton-miles moved on the river.

Miscellaneous Freight.—The savings per ton increased from 45 cts in 1925 to 66 cts in 1934. The savings per ton-mile decreased from 3.3 to 2.7 mills in the same period. This curious result is due to the increase in the volume of river-rail commerce moving on published rates (with considerably reduced unit savings), combined with a considerable increase in length of haul. The present trend of movement began in 1931. The saving of 3.3 mills is used prior to that year, and the saving of 2.7 mills thereafter.

Terminal Differentials.—The differentials used were determined from the best evidence available. In some cases the sources were confidential. On coal the value generally used was 5 cts per ton for loading, and 15 cts per ton for unloading. On unmanufactured iron and steel it was 15 cts per ton for both loading and unloading. On manufactured steel it was 25 cts per ton for loading and (on an average) 75 cts per ton for unloading. Incidentally, the unloading differential for manufactured iron and steel represents the entire cost of unloading, as the ultimate consignee nearly always takes rail delivery anyway, and, thus, all the cost of unloading from barges is an extra charge. On cement, the loading differential used was 15 cts per ton, and the unloading charge 75 cts per ton (except when it was delivered for use in riparian structures). On freight moved by common carrier, of course, the terminal differential is included in the published rate.

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PAPERS

SOLUTION OF TRANSMISSION PROBLEMS OF A WATER SYSTEM

By Ellwood H. Aldrich,[1] M. Am. Soc. C. E.

Synopsis

Some of the problems commonly experienced in the design of the transmission and distribution elements of a water system are solved in this paper which presents a rational method involving an expansion of the graphical solution[2] first suggested by the late John R. Freeman, Past-President and Hon. M. Am. Soc. C. E. A brief summary is given of the many variable and, from a practical standpoint, sometimes indeterminate, factors that must be considered in assembling the data upon which the design must be based. General cases are presented giving the solution of the more common problems encountered in design, with an illustration of the use of several of these general cases in the solution of a typical design problem.

Introduction

The design of the transmission and distribution facilities of a water system presents a complex and absorbing problem for one engaged in such a task. A mathematically accurate determination of the results to be expected in designing intricate grid systems or important reinforcements is impossible of attainment from a practical standpoint. When it is realized, however, that these elements of a water system commonly exceed 50% of the total value of such a system and that they materially influence the cost, as well as the adequacy of service, the reason becomes apparent why all the knowledge that is available should be utilized to secure an efficient design.

The opportunity to design a complete water transmission and distribution system of any magnitude is not often presented to an engineer. His work is usually concerned with providing extensions and reinforcements to existing systems. It is of primary importance for him, therefore, to determine the operating functions of the existing system as the basis for a determination of the necessity for, and the extent of, the most effective reinforcements.

Note.—Discussion on this paper will be closed in **February, 1938**, *Proceedings*.

[1] Civ. and San. Engr., Forest Hills, N. Y.

[2] *Journal*, New England Water Works Assoc., Vol. 7 (1892), p. 49.

Basic Data

Satisfactory service requires that a sufficient quantity of water shall be furnished to all parts of the system at pressures adequate for the requirements of the consumers under all conditions of their demands. To create such service proper reserves must be available in the form of pumping, transmission, and distribution capacity, often supplemented by storage, to meet all reasonable contingencies.

The more important basic data required by the designer before undertaking the solution of the hydraulics of a problem are: (*a*) The pressure and flow requirements under the varying rates to be expected in the respective parts of the system; (*b*) the topography of the service areas; (*c*) the pumping arrangements and pump characteristics in other than gravity systems; (*d*) the location, quantity, and elevation of storage; and (*e*) the condition of the pipe system in so far as the carrying capacities of its many elements are concerned.

Williams-Hazen Formula.—Many formulas have been developed to present the results of the flow of water in pipes. The Williams-Hazen formula has become more generally used than any other for water-works conditions because of the ease of its application with the aid of tables,[3] or by the use of a special slide-rule. It gives acceptable results provided the coefficient is carefully determined. This formula is:

$$V = C\,R^{0.63}\,S^{0.54}\,0.001^{-0.04} \quad (1)$$

in which V = velocity of flow, in feet per second; C = coefficient, dependent on the interior conditions of the pipe (sometimes called the coefficient of roughness); R = the hydraulic radius or the area of the pipe divided by the wetted perimeter, in feet; and, $S = \frac{H}{L}$, the slope of the hydraulic gradient or the head loss, in feet per thousand feet. In a closed pipe of constant diameter, with flow under pressure, $R^{0.63}$ becomes a constant, so that,

$$V = C\,K\,S^{0.54} \quad (2)$$

In other words the velocity, and, therefore, the quantity flowing, in a given size of pipe, vary directly as the coefficient, C, and as the 0.54 power of the hydraulic gradient.

Therefore, several fundamental relations result: With C constant, V varies as $S^{0.54}$, or S varies as $V^{1/0.54}$; with V constant, S varies as $C^{1/0.54}$, or C varies as $S^{0.54}$; and, with S constant, V varies directly as C.

In using the flow tables based upon this formula, which were compiled by the late Gardner S. Williams and Allen Hazen, Members, Am. Soc. C. E.,[3] it is necessary to have some convenient means of interpolating between the standard flows, sizes of pipe, losses of head, and coefficients set forth therein. By plotting the standard flows as determined from the Williams-Hazen tables, or by slide-rule, on logarithmic scale, and, similarly, by plotting S against $S^{0.54}$ and $S^{1/0.54}$, as shown in Fig. 1, the interpolation and conversion of the varying factors of

[3] "Hydraulic Tables," by the late Gardner S. Williams and Allen Hazen, Members, Am. Soc. C. E., John Wiley & Sons.

the formula are facilitated. With this formula as a base, it is possible to make all computations by the use of Fig. 1.

Solution of General Problems

General.—Specific problems usually involve as a first step the simplification of intricate arrangements of many pipe lines and their combination into one or more equivalent lines. In making these combinations, several general cases

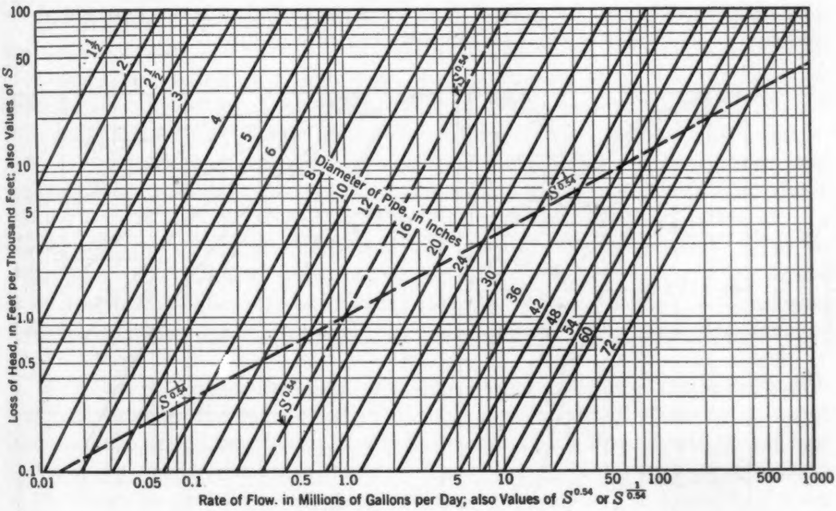


FIG. 1.—FRICTION LOSS BY EQUATION (1)

arise in almost every problem. These will be illustrated by the following general cases: (I) Pipe connected in series; (II) pipe connected in parallel and in series; (III) "take-outs" and "put-ins"; (IV) compound storage; (V) pipe connected in parallel with intermediate cross-connections; and, (VI) pipe connected in grids.

It is not the intent to imply that a graphical solution of the simpler combinations must be resorted to; neither is it always possible or necessary to take the time, or to make the effort, to achieve an accurate combination of many complex lines. Most combinations can be made by slide-rule computations without graphical help, but in certain cases much better progress can be made by using a graphical solution than is possible by trial and error, or by mathematical methods, and familiarity with the graphical representations of the simpler combinations is a prerequisite to an easy solution of the more complex systems.

The term, "hydraulic grade," is used in this paper to denote the piezometric pressure in the pipe or conduit at the point in question. Although in most of the cases discussed herein the coefficient has been assumed as 100, in order to permit the use of Fig. 1 without further computation, the special slide-rule permits solutions using actual coefficients with equal ease. It is also apparent

that the graphical method described herein is applicable to other formulas expressing the results of flow of water in pipes.

Case I.—Pipe Connected in Series.—Fig. 2 is a graphical representation of the loss of head under varying rates of flow through a pipe line consisting of several pipes of different diameter connected in series. In such a line the total head loss for a given flow is a summation of the loss of head in each section for that flow. With abscissas representing the rate of flow and ordinates the loss of head under those rates of flow, curves are constructed for each section of pipe. The total loss of head for several pipe lines connected in series consists of the addition of ordinates of each section for each rate of flow.

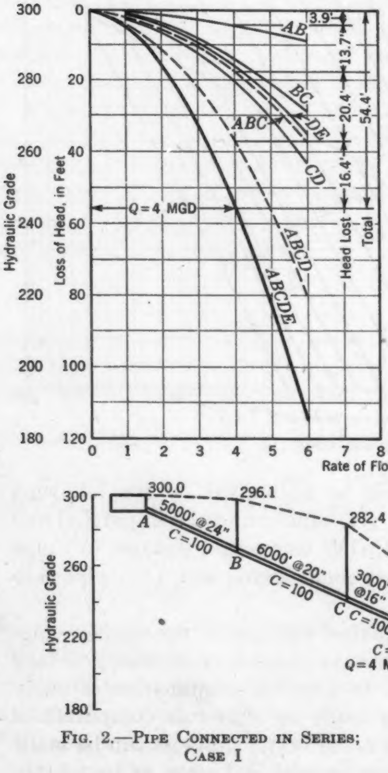


FIG. 2.—PIPE CONNECTED IN SERIES; CASE I

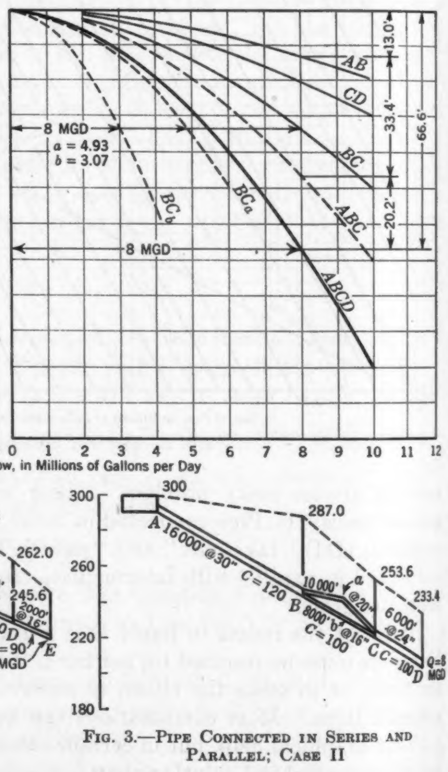


FIG. 3.—PIPE CONNECTED IN SERIES AND PARALLEL; CASE II

Case II.—Pipe Connected in Parallel and in Series.—Fig. 3 is a graphical representation of the loss of head under varying rates of flow through three sections of pipe, one of the sections consisting of two pipes connected in parallel. The graphical combination of pipe connected in parallel is effected by adding the abscissas of each pipe at each loss of head, since the loss of head through a given section consisting of several pipes connected in parallel is the same in each pipe and the total flow in the section is the sum of the flows in each pipe. Having determined the equivalent pipe curve for the section of pipes connected

in parallel, the three sections of pipe connected in series are combined as illustrated in Case I.

Cases I and II are simple combinations of pipe easily solved with the aid of the hydraulic slide-rule. However, the graphical combination of such pipe arrangements greatly facilitates solutions in more complex combinations, involving consumption at points along the line, the introduction of water from storage or from other sources, and those combinations of pipe for which several factors are dependent upon each other.

The graphical solution of these more complex systems requires the moving of the quantity-head loss curves to represent the effect of variation in elevation of storage, the putting in or taking out of quantities of water, and the flow and pressure characteristics of centrifugal pumps. These solutions are facilitated by a graphical representation of the quantity-head loss relation which can be moved easily to represent the variations in conditions. Convenient transparent media for such graphs are prepared in the form of small rectangles about 4 in. by 6 in. in size, cut from thin celluloid, processed or coated on one side to aid the drawing of the quantity-head loss curves and their removal when no longer required. Careful trimming of the edges aids accurate placing in the proper position. They are sufficiently rigid to lie flat and stay in place, and their transparency renders possible the use of several cards superimposed one upon another.

Case III.—"Take-Outs" and "Put-Ins."—Fig. 4(*a*) is a graphical representation showing the taking out or putting in of a certain quantity of water from, or into, one of two mains connected in parallel. The quantity-head loss curve, *A E C*, representing the flow in Main *A E C*, is drawn upon the right of the *Y*-axis as shown. With no "take-out" or "put-in" at Point *B*, Curves *A B* and *B C*, representing the flow conditions in Mains *A B* and *B C*, are drawn upon the left of the *Y*-axis and combined as in any two pipes connected in series, as illustrated in Case I. A rate of flow of 8 mgd at Point *C* results in a loss of head of 59.8 ft, 4.3 mgd flowing through Main *A B C* and 3.7 mgd through Main *A E C*.

With a "take-out" at the rate of 4 mgd at Point *B*, Curve *A B* is moved to the right a distance equivalent to 4 mgd on the *X*-axis, as shown in Curve *A B''*, before combination with Curve *B C* to form the curve, *A B* C^T. In extending Curve *A B* C^T to the right of the *Y*-axis, flow in Pipe *B C* reverses in direction, causing Curve *B C* to assume the position, *B C'*, which is used in combining the two curves for flows in Mains *A B* and *B C*. With a flow from Point *C* at the rate of 8 mgd and a "take-out" of 4 mgd at Point *B*, the loss of head from Points *A* to *C* is 84.5 ft, 4.5 mgd flowing through Main *A E C*, 7.5 mgd through Main *A B*, and 3.5 mgd through Main *B C*. By following vertically up from the intersection of this quantity with Curve *A B* C^T to the intersection with Curve *A B''* the loss of head in Main *A B* is found to be 58.5 ft.

The intersection of Curve *A B* C^T with the *Y*-axis gives the loss of head in the system at which reversal of flow in the main, *B C*, occurs. With any flow at Point *C* less than is given for this loss of head, the flow through Main *A E C* exceeds the quantity taken out at Point *C* and the remainder flows toward Point *B* to satisfy the "take-out" there. In other words, the scaled rate of

flow between Curves $A\,B\,C^T$ and $A\,E\,C$ represents the draft at Point C; that between $A\,B\,C^T$ and the Y-axis represents the flow back from Point C to Point B and the total flow through Main $A\,E\,C$ is the sum of the two, or the scaled distance from Curve $A\,E\,C$ to the Y-axis. The ordinate of the intersection of Curves $A\,B\,C^T$ and $A\,E\,C$ measures the loss of head from Points A to C, 6.5 ft, at which there can be no flow out of Point C if the "take-out" of 4 mgd at Point B is satisfied.

If 4 mgd is assumed to be put into the system at Point B, Curve $A\,B$ must be moved to the left of the Y-axis, opposite to its position for a "take-out", as Curve $A\,B'$, before combining with Curve $B\,C$ to form Curve $A\,B\,C^P$. With an 8-mgd rate of flow at Point C and a "put-in" of 4 mgd at Point B, the loss from Points A to C is 47.3 ft, causing a flow through Main $A\,E\,C$ of 3.3 mgd, through Main $A\,B$ of 0.7 mgd, and through Main $B\,C$ of 4.7 mgd. Following vertically up from the intersection of this total rate of flow with Curve $A\,B\,C^P$ to Curve $A\,B'$, the loss of head in Main $A\,B$ is 1.2 ft and, similarly, in Main $B\,C$ the loss is 46.1 ft. From these curves, the quantity-head loss relations in all pipes are established for flows at Point C under the three conditions assumed at Point B, namely, no "take-out" or "put-in", a "take-out" of 4 mgd, and a "put-in" of 4 mgd.

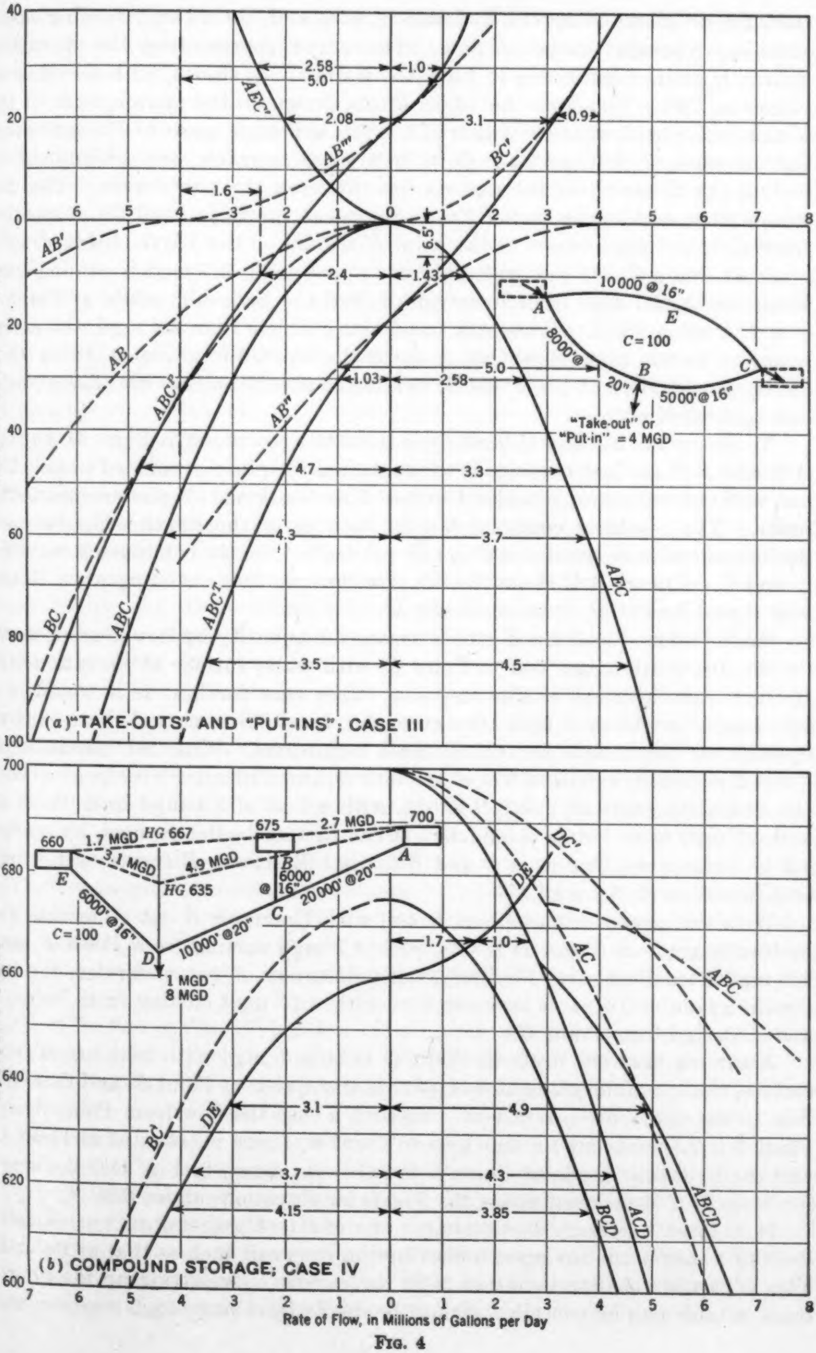
Another use of the curves in Fig. 4(a) can be made for reservoirs at Points A and C, capable of putting out or taking in the quantities of water required to balance the system without appreciable changes in elevations. A "take-out" of 4 mgd at Point B, when the elevation of Reservoir C is 30 ft below Reservoir A, causes a flow at the rate of 5 mgd from Points A to B, 1 mgd from Points B to C, and 2.58 mgd through Main $A\,E\,C$, or a total of 7.58 mgd flowing out of Reservoir A and 3.58 mgd flowing into Reservoir C. The hydraulic grade at Reservoir C, of course, is 30 ft below that at Reservoir A, and at Point B it is 27.5 ft below that at Point A, and 2.5 ft above that at Point C.

If Reservoir C is 20 ft above Reservoir A, under a similar "take-out" at Point B, there is a flow from Point C to Point A through Main $A\,E\,C$ of 2.08 mgd, a flow of 3.1 mgd from Points C to B, and one of 0.9 mgd from Points A to B, or a total flow of 5.18 mgd leaving Reservoir C and of 1.18 mgd entering Reservoir A. The hydraulic grades are determined similarly as described.

With a 4-mgd "put-in" at Point B and the elevation of Reservoir C 10 ft below Reservoir A, a flow of 1.43 mgd is caused from Point A to Point C in Main $A\,E\,C$, 1.6 mgd from Point B to Point A, and 2.4 mgd from Point B to Point C, or 3.83 mgd flowing into Reservoir C and 0.17 mgd into Reservoir A.

With a similar "put-in" at Point B, and Reservoir C 30 ft above Reservoir A, the resulting flow from Point C to Point A in Main $A\,E\,C$ is 2.58 mgd, from Points B to A, 5 mgd, and from Points C to B, 1 mgd, or 7.58 mgd flowing into Reservoir A and 3.58 mgd flowing out of Reservoir C.

Case IV.—Compound Storage.—Fig. 4(b) is a graphical solution of the losses of head under varying rates of flow from a system of supply or distributing reservoirs. With three reservoirs as shown, two of the reservoirs and their transmission mains are first combined into one source. Points A and B represent two supply reservoirs or intakes not subject to appreciable variations in levels, one situated at Elevation 700, and the other at Elevation 675. In this



Loss of Head, in Feet
10 000 @ 16"
8000'@ 20"
C = 100
5000'@ 16"
"Take-out" or "Put-in" = 4 MGD
(a) "TAKE-OUTS" AND "PUT-INS"; CASE III
Hydraulic Grade
660
1.7 MGD
HG 667
675
2.7 MGD
700
3.1 MGD
4.9 MGD
HG 635
6000' @ 16"
20 000' @ 20"
6000' @ 16"
C = 100
10 000' @ 20"
1 MGD
8 MGD
(b) COMPOUND STORAGE; CASE IV
Rate of Flow, in Millions of Gallons per Day

FIG. 4

case, the ordinates represent hydraulic grades and, as in the preceding cases, abscissas represent rates of flow. The curve representing the quantity-hydraulic grade relationship in Line *A C* is drawn, as shown, with zero flow at Elevation 700. The curve for Main *B C* is drawn on the opposite side of the *Y*-axis with zero flow at Elevation 675. The hydraulic grade at Point *C* under various rates of flow, and the flows from each reservoir, are determined by scaling the distance for the desired flow between the two curves. The distances from each curve to the *Y*-axis represent the flows from the respective reservoirs, and the location of the point of scaling on the *Y*-axis, the hydraulic grade at Point *C*. If the draft at Point *C* is 6 mgd, 3.7 mgd is coming from Reservoir *A*, 2.3 mgd from Reservoir *B*, and the hydraulic grade at Point *C* is at Elevation 660.5. If the draft at Point *C* is less than 2.9 mgd, the entire quantity comes from Reservoir *A* since the hydraulic grade at Point *C* is above the elevation of Reservoir *B*, tending to produce a flow into Reservoir *B* unless checked off.

To determine the loss of head from these two reservoirs at Point *D*, Curves *A B* and *B C* are first combined, as illustrated for pipes connected in parallel, and to this combination is added Curve *C D* as illustrated for pipes connected in series. The resulting curve, *A B C D*, then gives the relationship between drafts and hydraulic grades at Point *D* and drafts from the combined Reservoirs *A* and *B*. Curves *A C D* and *B C D* give the conditions existing when Reservoir *B* and Reservoir *A*, respectively, are shut off.

On the other side of the *Y*-axis is drawn the quantity-hydraulic grade curve for the distribution reservoir at Point *E*, with water surface at Elevation 660. If the water elevation of this reservoir varies considerably, as it would in a stand-pipe, or elevated tank, the curve for Line *DE* is moved downward or upward on the *Y*-axis to satisfy these conditions. With the elevation at Point *E* constant, a draft of 8 mgd at Point *D*, and with all reservoirs in service, the hydraulic grade at Point *D* is 635, with a flow of 3.1 mgd from Point *E*, and 4.9 mgd from Points *A* and *B*. It can further be determined, by scaling 4.9 units between Curves *A C* and *B C*, that Reservoir *B* furnishes 1.6 mgd and Reservoir *A*, 3.3 mgd.

With the same draft at Point *D* and with Reservoir *B* out of service, the hydraulic grade at Point *D* is 623, with 3.7 mgd coming from Point *E*, and 4.3 mgd from Point *A*. Similarly, with Reservoir *A* out of service, the hydraulic grade at Point *D* becomes 616, with 4.15 mgd coming from Point *E* and 3.85 mgd from Point *B*.

Assuming that the draft at Point *D* is only 1 mgd with both supplies in service, the hydraulic grade at this point is above that at Point *E*, and there is a flow to the distribution reservoir. In such a case the flow from Point *A* and Point *B* is 2.7 mgd, but 1.7 mgd goes to Point *E*, 1 mgd is taken off at Point *D*, and the hydraulic grade at Point *D* is 667. As shown in Fig. 4(*b*) the curve for Main *D E* is reversed about the *Y*-axis for elevations above 660.

It is often necessary to determine the effect of a sustained but variable draft of a maximum day upon a distribution reservoir such as that at Point *E*. This is capable of determination from the curves. By setting up the hourly rates, a table can be compiled giving the drafts from or to each reservoir and

the hydraulic grades at the desired points. This can be further complicated when the storage in any of the reservoirs varies in elevation by any appreciable extent.

A study such as this is particularly important in determining the size and elevation of storage. A sufficiently accurate determination can be made upon an hourly basis by converting the quantity drawn from a stand-pipe or tank to gallons, and from the diameter of the tank, converting this to drop in water elevation, in feet. During the next hour the curve must be moved down by the quantity of the drop. This will result in less water being drawn from the tank or the stand-pipe. This approximation can be continued, hour by hour, until, the peak loads past, or the lowering level in the tank (or more generally a combination of the two), the hydraulic grades produce a flow toward the tank and it begins to fill. In this manner curves can be drawn for almost any condition, giving the draft and hydraulic grade at a certain point, the flow from or to a stand-pipe or tank, the flows from the sources of supply, and the hydraulic grade, or its equivalent, the elevation of water in the tank.

Case V.—Pipes Connected in Parallel with Intermediate Cross-Connections.—Fig. 5(*a*) represents the flow conditions and losses of head through two pipes connected in parallel with an intermediate cross-connection (see Fig. 5(*b*)). The arrangement of the pipes and the construction of the individual quantity-head loss curves are self-explanatory. In all examples involving cross-connected pipe, the cross-connection is in effect a "take-out" of one pipe and a "put-in" to another of an unknown quantity, dependent upon the carrying capacities of the several pipes.

Since the hydraulic grades at the junction of Pipes (1) and (2) are the same, curves for these pipes are drawn, as shown, on each side of the Y-axis. It is unnecessary to have a definite flow through the system in order to convert it into an equivalent pipe; therefore, in Pipe (2) a loss of head of 30 ft is assumed. This produces a flow of 4.22 mgd through Pipe (2). The remaining unknowns are h_1, h_3, h_4, h_5, Q_1, Q_3, Q_4, and Q_5. It is known, however, that at all junction points the hydraulic grades are the same in each pipe, and the quantity flowing to and from each junction is in balance. Usually, by inspection, it is possible to determine the direction of flow in interior pipes, and curves for these pipes can be laid off, either to the right or to the left, with zero flow at the assumed head loss to the junction in question. In this case, from the lengths and sizes of pipe, it is evident that flow in Pipe (3) is from Point C to Point B, that a part of the flow in Pipe (2) is taken out at the cross-connection, leaving a smaller quantity to flow through Pipe (4). This "take-out" becomes a "put-in" at the other end of the cross-connection and is added to the flow in Pipe (1) to produce the flow in Pipe (5).

Curves (4) and (5) are traced on a movable card from Curves (4)′ and (5)′, which are set up anywhere. It is known that the losses of head to the ends of Pipes (4) and (5) are the same; therefore, they are drawn together as shown in Fig. 5(*a*). From inspection, it is seen that Curve (4) must intersect Curve (2) at Point a, at the same loss of head as the beginning of Curve (3). Similarly, Curve (5) must intersect Curve (1) at Point b. The card on which Curves (4) and (5) are drawn must be adjusted, therefore, so that Curve (4) intersects

Curve (2) at Point *a*, Curve (5) intersects Curve (1) at Point *b*, and the *Y*-axis of Curves (4) and (5) must intersect Curve (3) at a point, *c*, with the same loss of head and, therefore, in the same horizontal plane in Fig. 5(*a*), as Point *b*. Having adjusted the card in this manner, the following conditions have been met:

$$H = h_1 + h_5 = h_2 + h_4 = h_2 + h_3 + h_5 = h_1 - h_3 + h_4 \ldots\ldots (3)$$

or, in feet, 72.8 = 32.4 + 40.4 = 30.0 + 42.8 = 30.0 + 2.4 + 40.4 = 32.4 − 2.4 + 42.8; and,

$$Q = Q_1 + Q_2 = Q_4 + Q_5 = Q_2 - Q_3 + Q_5 = Q_1 + Q_3 + Q_4 \ldots\ldots (4)$$

or, in million gallons daily, 8.12 = 3.9 + 4.22 = 3.12 + 5.0 = 4.22 − 1.1 + 5.0 = 3.9 + 1.1 + 3.12.

In Fig. 5(*b*) is shown a further step of the preceding problem with an additional interior connecting pipe. Similar procedure is adopted for its solution. Curves (1) and (2) are drawn as before and a loss of head of 30 ft is assumed in Pipe (2) giving a flow of 4.2 mgd. Since Pipes (3) and (4) begin at the same hydraulic grade, Pipes (5) and (6) and Pipes (7) and (8) end at the same grade, these sets of curves are drawn together, and there are three cards to be adjusted.

Curves (3) and (4) must start with the loss of head that occurs in Pipe (2), which, by assumption, is 30 ft. Similarly, from the condition of a balanced hydraulic grade at each junction, Curves (3) and (4), (5) and (6), and (7) and (8) must be adjusted so that the respective intersections, Points *a* to *k*, are as shown. Curve (3) can be drawn the same as in the previous example, but here it is as convenient to combine it with Curve (4) on the same movable card. There are many ways to arrange the curves for quantity-head loss. In these illustrations curves concave downward indicate flow away from the inlet, and curves concave upward indicate flow toward the outlet or flows from, or to, the apices of the curves. The interior curves may be considered either way as long as the fundamental relationships hold.

In these examples, the loss of head must be the same by any route connecting two points; and the quantity flowing through any straight section cut entirely through the system, must be the same, due regard being given to the direction of flow. The total head loss in the last system of pipes is 103.9 ft for a flow of 8 mgd. An equivalent pipe would be computed as follows: $\frac{103.9}{24.3}$ = 4 280 ft of 16-in. pipe, C = 100.

In the last illustration it is interesting to determine how much the cross-connecting pipes reduce the total loss of head. With two 16-in. pipes connected in parallel, one 20 000 ft, and the other 13 000 ft, long, eliminating the cross-connecting pipes, the equivalent pipe is 4 400 ft of 16-in. pipe, C = 100. The head loss is 24.3 × 4.4 = 106.9 ft, or the saving in head is only 3 ft. The conclusion is that pipes connecting points of approximately equal pressure are of little value in reducing the friction loss. As these pipes approach a parallel connection with the main pipes or connect points of greater pressure difference, their value in reducing losses or increasing flows becomes greater.

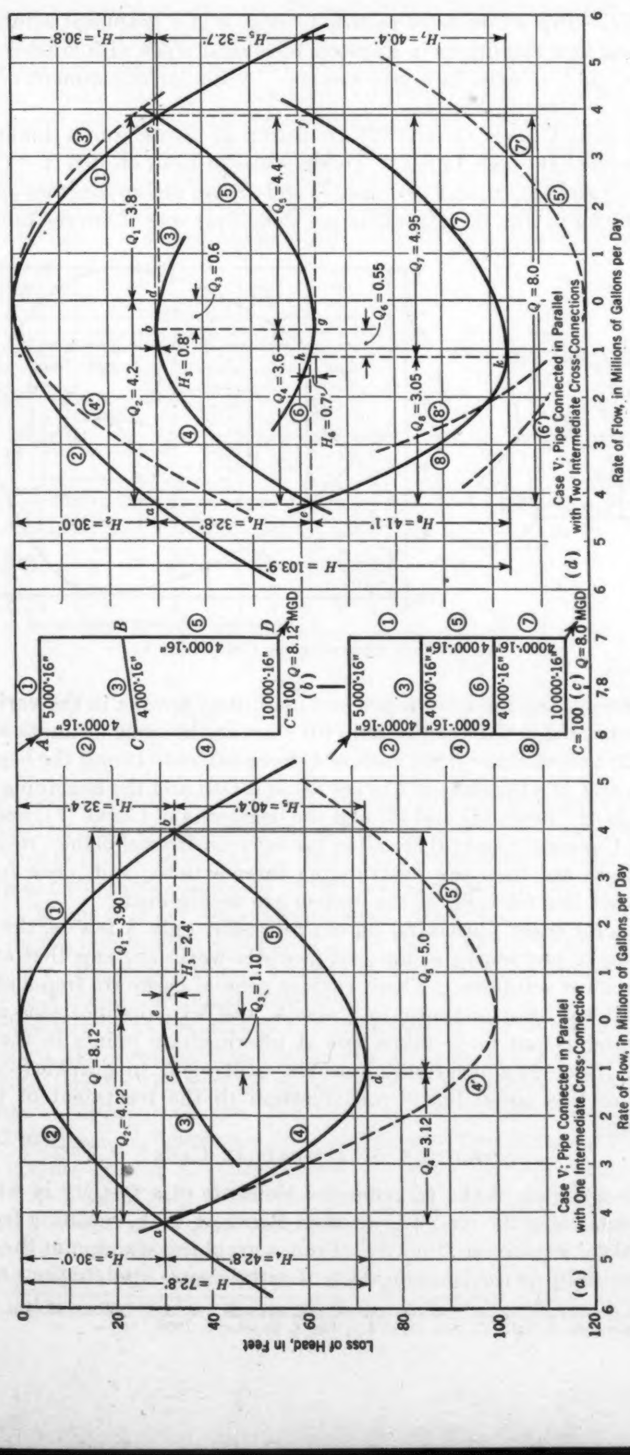

Loss of Head, in Feet
Rate of Flow, in Millions of Gallons per Day
(a) Case V; Pipe Connected in Parallel with One Intermediate Cross-Connection
(b) C=100 Q=8.12 MGD
(c) C=100 Q=8.0 MGD
(d) Case V; Pipe Connected in Parallel with Two Intermediate Cross-Connections
H=72.8'
H₂=30.0'
H₄=42.8'
H₁=32.4'
H₅=40.4'
H₃=2.4'
Q=8.12
Q₁=3.90
Q₂=4.22
Q₃=1.10
Q₄=3.12
Q₅=5.0
5000'-16"
4000'-16"
10000'-16"
6000'-16"
H=103.9'
H₁=30.8'
H₅=32.7'
H₇=40.4'
H₈=41.1'
H₆=0.7'
H₃=0.8'
Q₁=3.8
Q₂=4.2
Q₃=0.6
Q₄=3.6
Q₅=4.4
Q₆=0.55
Q₇=4.95
Q₈=3.05
Q=8.0

FIG. 5

Case VI.—Pipes Connected in Grids.—Fig. 6 is a graphical solution of the pressure and flow conditions in a system of several pipes with cross-connections both ways, such as exist in a grid system. A similar adjustment of curves is required as in Case V, Fig. 5.

In this case, Curves (1) and (2) are drawn as before, and a quantity of 7.5 mgd is assumed through Line (2), giving a loss of head of 36.6 ft. Curves (3) and (10), (4) and (5), (6) and (9), and (7) and (8) are drawn together on movable cards as shown in Fig. 6. By adjusting these four sets of curves to obtain the

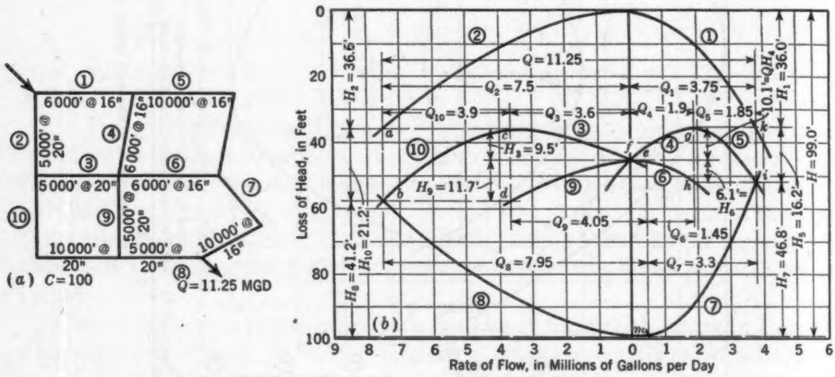


FIG. 6.—PIPE CONNECTED IN GRID, CASE VI

proper intersections, the head losses and quantities flowing in the various pipes can be determined. Curves (3) and (10) start at the same ordinate as the end of Curve (2) and, similarly, the ends of Curves (3) and (4) and the beginning of Curves (6) and (9); the ends of Curves (9) and (10) and the beginning of Curve (8); the ends of Curves (5) and (6) and the beginning of Curve (7); and, finally, the ends of Curves (7) and (8) are all in the same horizontal planes, respectively. Points *a* to *m* are then the determining intersections, and, once found, the quantity-head loss relations of the system are determined.

The general cases illustrated herein represent only a few of the simplest combinations of problems encountered in water-works systems that are adaptable to graphical solutions. These various general cases are frequently found in combinations. For instance, in Cases V and VI, complications are introduced if water is put in or taken out at intermediate points in the systems illustrated. Such complications, however, still yield to graphical solutions. The scope of this paper limits consideration to the treatment of the more general cases.

SOLUTION OF A GENERAL CASE

Fig. 7 is a sketch of the transmission elements of a system[4] in which it is desired to determine the loss of head from Points *A* to *X*, resulting from a fire flow of 1 500 gal per min at Point *X*. From a graphical standpoint the problem consists essentially in the simplification of certain pipe combinations to reduce

[4] "Simplified Analysis of Flow in Water Distribution Systems," by J. J. Doland, M. Am. Soc. C. E., *Engineering News-Record*, Vol. 117, No. 14, p. 475, Fig. 1, October 1, 1936.

the system into a problem similar to Case V; that is, pipes connected in parallel with an intermediate cross-connection.

To use the graphical method, it is necessary to have definite flows through the system, which may be determined, however, upon a percentage basis. Since the use of the hydraulic slide-rule simplifies the computations if flows are expressed in terms of million gallons per day, this basis is adopted herein and the fire flow of 1 500 gal per min, which forms 45% of the total, becomes 2.16 mgd of a total of 4.8 mgd. The various "take-outs" at points along the mains are shown in Fig. 7 in percentages, which are converted to million gallons per day in the computations.

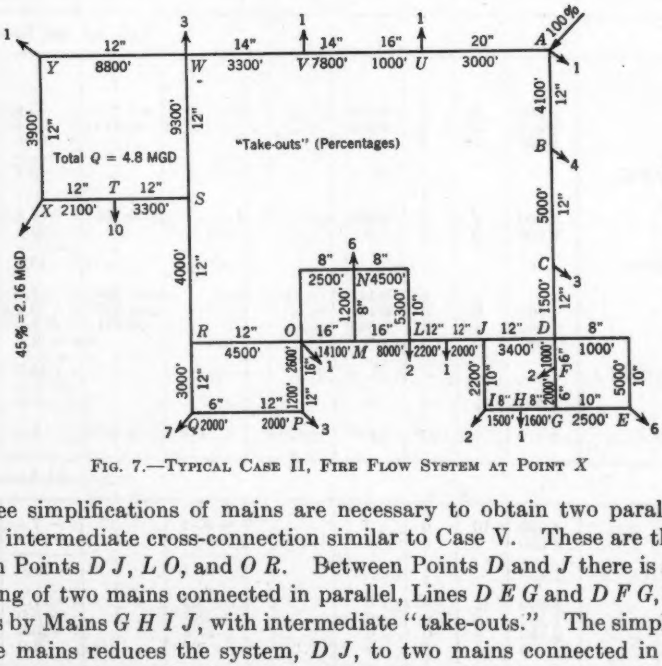


FIG. 7.—TYPICAL CASE II, FIRE FLOW SYSTEM AT POINT X

Three simplifications of mains are necessary to obtain two parallel pipes with an intermediate cross-connection similar to Case V. These are the mains between Points D J, L O, and O R. Between Points D and J there is a system consisting of two mains connected in parallel, Lines D E G and D F G, followed in series by Mains G H I J, with intermediate "take-outs." The simplification of these mains reduces the system, D J, to two mains connected in parallel, which requires the application of Case II to determine the losses of head from Points D to J under varying rates of flow.

Since the flow in any section of the main system will be unknown until the final solution, the computations and graphs must provide for a range of flows sufficient to include the various possibilities, with provision for subsequent "take-outs." Table 1(a) and Fig. 8(a) represent the computations and graphical analyses to determine the head-capacity relationship of the mains between Points D and J. The compound mains between Points L and O are similar to Case V with an intermediate "take-out," and the head-capacity relationship is determined as previously described. The computations and graphical representations are given in Table 1(b) and Fig. 9. Similarly, the mains between Points O and R, two mains connected in parallel, with intermediate "take-outs," are solved as given in Table 1(c) and Fig. 8(b).

TABLE 1.—LOSS OF

Lines (see Fig. 7)	DIMENSIONS		"TAKE-OUT"		QUANTITIES, IN MILLION GALLONS				
					−1.0		−0.5		−0.25
	Length, in feet	Diameter, in inches	In percentages	In million gallons daily	Quantity, in million gallons daily	Loss of head, in feet	Quantity, in million gallons daily	Loss of head, in feet	Quantity, in million gallons daily
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
									(*a*) LOSS OF HEAD
D G H I J									
D E G									
D E	1 000	8	6	0.288	....	....	−0.212	−0.9	0.038
	5 000	10	6	0.288	....	....	−0.212	−1.4	0.038
E G	2 500	10	0	0	....	....	....	−3.5	....
Total in *D E G*	....	..	..	....	....	....	....	−5.8	....
D F G									
D F	1 000	6	2	0.096	....	....	−0.404	−11.5	−0.154
F G	2 000	6	0	0	....	....	....	−34.0	....
Total in *D F G*	....	..	..	....	....	....	....	−45.5	....
D G	....	..	3	0.144	....	....	−0.356	− 1.1	−0.106
G H	1 600	8	3	0.144	....	....	−0.356	− 3.6	−0.106
H I	1 500	8	2	0.096	....	....	−0.404	− 4.2	−0.154
I J	2 200	10	0	0	....	....	....	− 3.1	....
Total in *D G H I J*	....	..	..	....	....	....	....	−12.0	....
D J	3 400	12	0	0	....	....	....	−2.0	....
Total, *D* to *J*	....	..	20*	0.96*	−0.04†	0.3	0.46†	2.5	0.71†
									(*b*) LOSS OF HEAD
L N	5 300	10	0	0	....	−27.1	....	− 7.5	....
	4 500	8	0	0	....	−68.0	....	−18.9	....
Total in *L N*	....	..	..	....	....	−95.1	....	−26.4	....
L M	8 000	16	0	0	....	− 4.2	....	− 1.2	....
M N	1 200	8	0	0	....	−18.1	....	− 5.0	....
N O	2 500	8	0	0	....	−37.8	....	−10.5	....
M O	14 100	16	0	0	....	− 7.3	....	− 2.0	....
Total, *L* to *O*‡	....	..	11‖	0.528‖	−0.472¶	− 1.4	0.028¶	0.6	0.278¶
									(*c*) LOSS OF HEAD
O P Q R									
O P	2 600	16	10	0.48	−0.52	− 0.4	−0.02	0.0	0.23
	1 200	12	10	0.48	−0.52	− 0.8	−0.02	0.0	0.23
P Q	2 000	12	7	0.336	−0.664	− 2.0	−0.164	−0.1	0.086
	2 000	6	7	0.336	−0.664	−57.5	−0.164	−4.2	0.086
Q R	3 000	12	0	0	....	− 6.3	....	−1.7	....
Total in *O P Q R*	....	..	..	....	....	−67.0	....	−6.0	....
O R	4 500	12	0	0	....	− 9.5	....	−2.6	....
Total, *O* to *R*	....	..	0[c]	0[c]	−1.0[d]	− 3.6	−0.5[d]	−0.6	−0.25[d]

* "Take-outs" from Points *J* to *S*. † The rate of flow at Point *J*. ‡ Corresponding rate of flow Points *O* to *S*. ¶ Rate of flow at Point *O*. [c] Corresponding rate of flow at Point *O* = 2.028 mgd. of flow at Point *R*. For − 1.5 mgd, the flow at Point *R* = − 1.5 mgd and the loss of head at Point Point *R* = 2.0 mgd.

HEAD IN MAINS

DAILY, FROM THE END OF THE LINE $A\ B \ldots S$

−0.25	0		0.25		0.50		1.0		1.5	2.0
Loss of head, in feet	Quantity, in million gallons daily	Loss of head, in feet	Quantity, in million gallons daily	Loss of head, in feet	Quantity, in million gallons daily	Loss of head, in feet	Quantity, in million gallons daily	Loss of head, in feet	Loss of head, in feet	Loss of head, in feet
(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)
IN MAINS D TO J (SEE FIG. 8(a))										
0.0	0.288	1.5	0.538	4.8	0.788	9.8	1.288	24.2	....	....
0.1	0.288	2.5	0.538	8.1	0.788	16.5	1.288	41.0	....	....
− 1.0	....	0.0	....	1.0	....	3.5	....	12.7	....	....
− 0.9	....	4.0	....	13.9	....	29.8	....	77.9	....	....
− 1.9	0.096	0.8	0.346	8.6	0.596	23.4	1.096	73.0	....	....
− 9.4	....	0.0	....	9.4	....	34.0	....	122.2	....	....
−11.3	....	0.8	....	18.0	....	57.4	....	195.2	....	....
1.0	0.144	5.2	0.394	12.0	0.644	20.7	1.144	46.0	....	....
− 0.4	0.144	0.7	0.394	4.3	0.644	10.7	1.144	31.0	....	....
− 0.7	0.096	0.3	0.346	3.2	0.596	8.7	1.096	27.0	....	....
− 0.9	....	0.0	....	0.9	....	3.1	....	11.2	....	....
− 1.0	....	6.2	....	20.4	....	43.2	....	115.2	....	....
− 0.5	....	0.0	....	0.5	....	2.0	....	7.1	....	....
4.2	0.96†	6.5	1.21†	9.1	1.46†	12.3	1.96†	19.7	28.5‡	38.0§
IN MAINS L TO O (SEE FIG. 9)										
− 2.1	....	0	....	2.1	....	7.5	....	27.1	57.5	....
− 5.2	....	0	....	5.2	....	18.9	....	68.0	144.3	....
− 7.3	....	0	....	7.3	....	26.4	....	95.1	201.8	....
− 0.3	....	0	....	0.3	....	1.2	....	4.2	8.8	15.0
− 1.4	....	0	....	1.4	....	5.0	....	18.1	38.5	66.0
− 2.9	....	0	....	2.9	....	10.5	....	37.8	80.3	137.5
− 0.6	....	0	....	0.6	....	2.0	....	7.3	15.5	26.4
1.9	0.528¶	4.0	0.778¶	6.3	1.028¶	9.5	1.528¶	18.0	29.0[a]	42.8[b]
IN MAINS O TO R (SEE FIG. 8(b))										
0.1	0.48	0.4	0.73	0.8	0.98	1.3	....	....	....	....
0.2	0.48	0.6	0.73	1.4	0.98	2.4	....	....	....	....
0.0	0.336	0.6	0.586	1.6	0.836	3.0	....	....	....	....
1.3	0.336	16.3	0.586	45.6	0.836	88.0	....	....	....	....
− 0.5	....	0	....	0.5	....	1.7	....	....	....	....
1.1	....	17.9	....	49.9	....	96.4	....	....	....	....
0.7	....	0.0	....	0.7	....	2.6	....	9.5	....	....
0.0	0.0[d]	0.7	0.25[d]	2.1	0.5[d]	4.4	1.0[d]	11.0	20[e]	31[f]

at Point J = 2.46 mgd. § Corresponding rate of flow at Point J = 2.96 mgd. ‖ "Take-outs" from [b] Corresponding rate of flow at Point O = 2.528 mgd. [c] "Take-outs" from Points R to S. [d] The rate R = − 8.7 ft. [e] Corresponding rate of flow at Point R = 1.5 mgd. [f] Corresponding rate of flow at

TABLE 2.—Loss of Head

Lines (see Fig. 7)	Dimensions		"Take-Out"		Quantities, in Million Gallons									
					−1.0		−0.5		−0.25		0		0.25	
	Length, in feet	Diameter, in inches	In percentages	In million gallons daily	Quantity, in million gallons daily	Loss of head, in feet	Quantity, in million gallons daily	Loss of head, in feet	Quantity, in million gallons daily	Loss of head, in feet	Quantity, in million gallons daily	Loss of head, in feet	Quantity, in million gallons daily	Loss of head, in feet
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
														(*a*) Circuit
A B	4 100	12	38	1.824	0.824	6.0	1.324	14.5	1.574	20.0	1.824	26.2	2.074	33.2
B C	5 000	12	34	1.632	0.632	4.5	1.132	13.2	1.382	19.0	1.632	26.0	1.882	33.8
C D	1 500	12	31	1.488	0.488	0.8	0.988	3.1	1.238	4.7	1.488	6.6	1.738	8.7
*D J**	..	..	20	0.96	−0.04	0.3	0.46	2.5	0.71	4.2	0.96	6.5	1.21	9.1
J K	2 000	12	20	0.96	−0.04	0.0	0.46	1.0	0.71	2.2	0.96	3.9	1.21	6.0
K L	2 200	12	19	0.912	−0.088	−0.1	0.412	0.9	0.662	2.2	0.912	3.9	1.162	6.1
L O†	..	..	11	0.528	−0.472	−1.4	0.028	0.6	0.278	1.9	0.528	4.0	0.778	6.3
O R‡	..	..	0	0	....	−3.6	....	−0.6	....	0.0	....	0.7	....	2.1
R S	4 000	12	0	0	....	−8.4	....	−2.3	....	−0.6	....	0.0	....	0.6
Total, Circuit *A B* to *S*	..	..	..	..	....	−1.9	....	32.9	....	53.6	....	77.8	....	105.9
														(*b*) Circuit
A U	3 000	20	5	0.24	....	....	....	....	....	....	0.24	0.0	0.49	0.1
U Y	1 000	16	4	0.192	....	....	....	....	....	....	0.192	0.0	0.442	0.1
	7 800	14	4	0.192	....	....	....	....	....	....	0.192	0.4	0.442	1.7
Y W	3 300	14	3	0.144	....	....	....	....	....	....	0.144	0.1	0.394	0.6
Total, Circuit *A U* to *W*	..	..	..	....	....	....	....	....	....	....	....	0.5	....	2.5
														(*c*) Line
S T	3 300	12	10	0.48	−0.52	−2.1	−0.02	0.0	0.23	0.5	0.48	1.8	0.73	3.9
T X	2 100	12	0	0	....	−4.4	....	−1.2	....	−0.3	....	0.0	....	0.3
Total, Line *S T X*	..	..	..	....	....	−6.5	....	−1.2	....	0.2	....	1.8	....	4.2
														(*d*) Line
W Y	8 800	12	1	0.048	....	....	....	....	....	....	0.048	0.1	0.298	2.0
Y X	3 900	12	0	0	....	....	....	....	....	....	....	0.0	....	0.6
Total, Line *W Y X*	..	..	..	....	....	....	....	....	....	....	....	0.1	....	2.6
														(*e*) Line
W S	9 300	12	0	0	....	....	....	....	....	....	....	0.0	....	1.5

* See Fig. 7. † See Fig. 8(*a*). ‡ See Fig. 9.

IN THE ENTIRE SYSTEM

DAILY, FROM THE END OF THE LINE

0.50		1.0		1.5		2.0		2.5		3.0	
Quantity, in million gallons daily	Loss of head, in feet	Quantity, in million gallons daily	Loss of head, in feet	Quantity, in million gallons daily	Loss of head, in feet	Quantity, in million gallons daily	Loss of head, in feet	Quantity, in million gallons daily	Loss of head, in feet	Quantity, in million gallons daily	Loss of head, in feet
(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)	(27)
A B TO *S*											
2.324	41.0	....	....	....	....	....	....	....	....	....	....
2.132	42.8	....	....	....	....	....	....	....	....	....	....
1.988	11.3	....	....	....	....	....	....	....	....	....	....
1.46	12.3	....	....	....	....	....	....	....	....	....	....
1.46	8.4	....	....	....	....	....	....	....	....	....	....
1.412	8.7	....	....	....	....	....	....	....	....	....	....
1.028	9.5	....	....	....	....	....	....	....	....	....	....
....	4.4	....	....	....	....	....	....	....	....	....	....
....	2.3	....	....	....	....	....	....	....	....	....	....
....	140.7	....	....	....	....	....	....	....	....	....	....
A U TO *W*											
0.74	0.3	1.24	0.8	1.74	1.5	2.24	2.3	2.74	3.4	3.24	4.6
0.692	0.3	1.192	0.7	1.692	1.4	2.192	2.2	2.692	3.2	3.192	4.4
0.692	3.9	1.192	10.6	1.692	20.4	2.192	33.0	2.692	48.4	3.192	66.4
0.644	1.4	1.144	4.2	1.644	8.2	2.144	13.4	2.644	19.8	3.144	27.2
....	5.9	....	16.3	....	31.5	....	50.9	....	74.8	....	102.6
S T X											
0.98	6.6	1.48	14.3	1.98	24.5	2.48	37.3	....	....	....	....
....	1.2	....	4.4	....	9.4	....	16.0	....	....	....	....
....	7.8	....	18.7	....	33.9	....	53.3	....	....	....	....
W Y X											
0.548	6.0	1.048	20.2	1.548	41.6	2.048	69.6	....	....	....	....
....	2.3	....	8.2	....	17.4	....	29.6	....	....	....	....
....	8.3	....	28.4	....	59.0	....	99.2	....	....	....	....
W S											
....	5.4	....	19.6	....	41.5	....	70.8	....	107.0	....	....

Having simplified the main, *A B . . . S*, the final computations and graph can be determined. Table 2 presents the computations upon which the curves in Fig. 10 are based. As previously described in Case V, for a total flow of 2.16 mgd at Point *X*, the loss of head is found to be 116.5 ft, and the quantities and

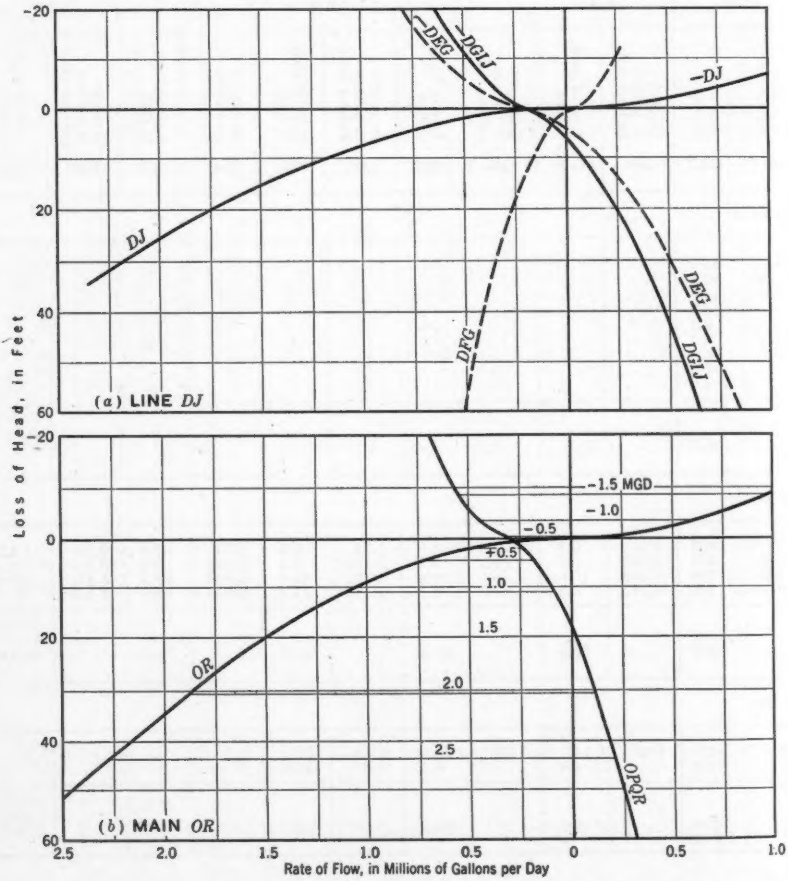


FIG. 8.—GRAPHS TO DETERMINE LOSS OF HEAD DUE TO FIRE FLOW AT POINT *X*

losses of head in the various main elements of the system will be as shown in Table 3. In explanation of the method of taking care of "take-outs," and to allow for the additional quantities through Lines *A W* and *A S* required for

TABLE 3.—QUANTITIES AND LOSSES OF HEAD IN MAIN ELEMENTS

Description	Total	*A W*	*A S*	*W S*	*W X*	*S X*
Quantity, in million gallons daily........	2.16	2.48	0.21	1.18	1.25	0.91
Loss of head, in feet....................	116.5	74.0	101.0	26.5	42.5	15.5

subsequent "take-outs," the curve for Line A W is moved to the right a distance equal to 1% of 4.8, or 0.048 mgd, and the curve for Line A S is moved to the left a distance equal to 10%, or 0.48 mgd, as shown in the dashed lines on Fig. 10.

The foregoing solution secures only one practical result—the loss of head in the system, under certain consumption requirements, with a definite fire flow at

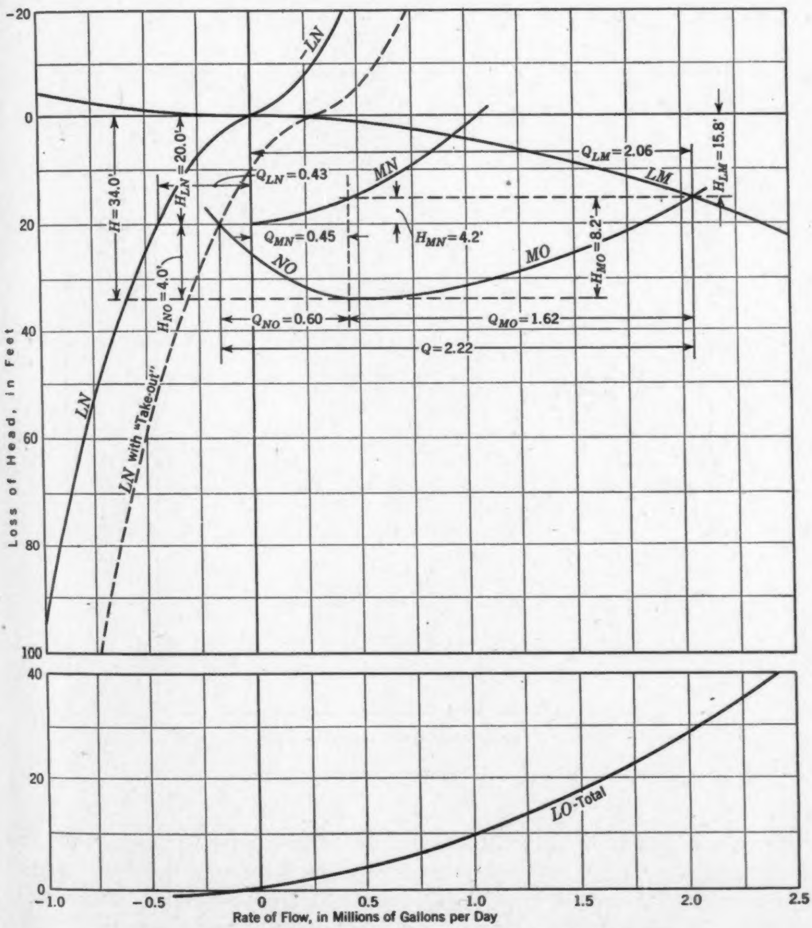


FIG. 9.—TYPICAL CASE II, LOSS OF HEAD, L TO O

Point C. It is true that from this solution other flow and head losses can be computed by the $S^{0.54}$-relationship, but in this conversion the intermediate "take-outs" vary in the same ratio, a condition that occurs rarely in practical studies. The consumption in a system bears no relationship to fire flows, or to other extraordinary requirements. To secure the loss of head under any other fire flow with the same "take-outs," or the loss of head with the same fire flow

under different consumption conditions, requires separate computations unless it is possible to construct a curve that will allow various flows to be taken through the system and the determination of losses of head without changing the quantities taken out. By obtaining various values of flow and corresponding head losses, a curve can be constructed from Fig. 10 which, the "take-outs" remaining the same, will give the head loss under any reasonable flow at Point X.

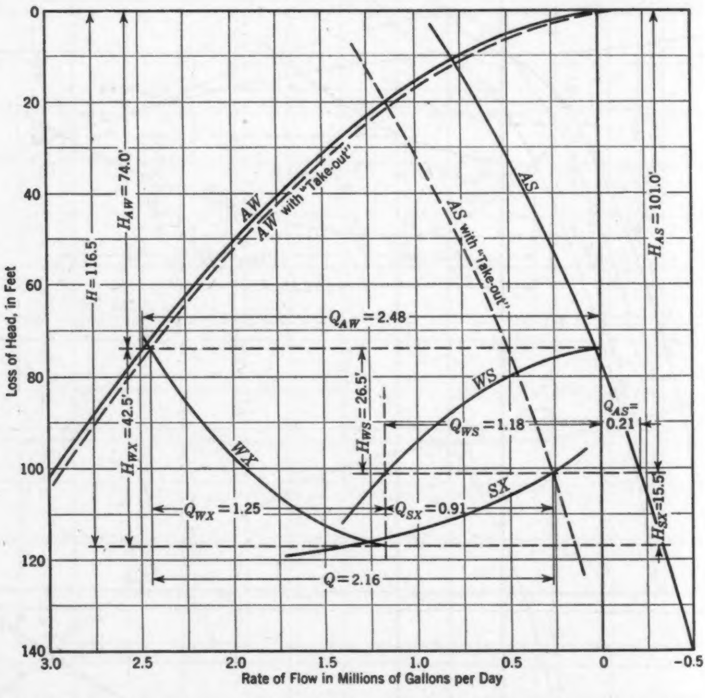


FIG. 10.—GRAPH TO DETERMINE LOSS OF HEAD IN MAIN AX DUE TO FIRE FLOW AT POINT X

From a practical viewpoint and to permit solutions for purposes other than the determination of the effect of a single fire flow (such as in the determination of pressures at Point X, or at any point, under various daily or hourly drafts, to calculate the heads under which pumps must operate, or to examine into the required capacity and height of storage), a slightly different analysis is of more value. For instance, let it be assumed that the rates of draft for ordinary domestic and industrial consumption vary from 1.5 to 3.0 mgd. If a series of capacity-head curves representing the loss of head under rates of 1.5, 2.0, 2.5, and 3.0 mgd are constructed, then, by interpolation, the loss of head under any flow and under varying rates of draft in the system can be secured directly.

To do this, four curves are constructed, representing losses of head through the system under rates of 1.5, 2.0, 2.5, and 3.0 mgd. In this analysis, the

percentage "take-outs" are revised, as shown in Table 4, divorcing the normal consumption in the system from any extraordinary demands at Point X.

TABLE 4.—REVISION OF "TAKE-OUTS"

Point	"Take-out" (see Fig. 7), in percentages	REVISED "TAKE-OUT," PERCENTAGES		Point	"Take-out" (see Fig. 7), in percentages	REVISED "TAKE-OUT," PERCENTAGES		Point	"Take-out" (see Fig. 7), in percentages	REVISED "TAKE-OUT," PERCENTAGES	
		Theoretical	Value used			Theoretical	Value used			Theoretical	Value used
(1)	(2)	(3)	(4)	(1)	(2)	(3)	(4)	(1)	(2)	(3)	(4)
A	1	1.82	1	*K*	1	1.82	1	*U*	1	1.82	2
B	4	7.27	7	*L*	2	3.63	4	*V*	1	1.82	2
C	3	5.45	5	*N*	6	10.91	11	*W*	3	5.45	5
E	6	10.91	11	*O*	1	1.82	2	*Y*	1	1.82	2
F	2	3.63	4	*P*	3	5.45	5	*Z*	45	....	2
H	1	1.82	1	*Q*	7	12.73	13	Total, Points *A* to *Z*	100	100.00	100
I	2	3.63	4	*T*	10	18.20	18				

Fig. 11 shows the head-capacity curves of the main lines which form the basis for the 2.5-mgd rate curve of Fig. 12. The latter shows four curves which give the head-capacity relationship for any flow from Point X under rates of 1.5, 2.0, 2.5, and 3.0 mgd, distributed through the system on the percentages shown on Table 4. By scaling the distances between the curves and the Y-axis, the losses of head for a flow of 2.0 mgd from Point X are determined, as follows:

Normal consumption, in million gallons per day	Loss of head, in feet
1.5	73.6
2.0	87.8
2.5	102.5
3.0	120.0

It is interesting to note in Fig. 12 that by scaling a distance on each rate curve equal to the same percentage of the total, a curve is constructed that may be termed a percentage curve, which can be extended, by the $S^{0.54}$-relationship, to any value. For instance, in the typical example, Fig. 7, the normal consumption in the system is 2.64 mgd. The extraordinary fire demand of 2.16 mgd is 81.7% of the normal consumption. Therefore, if a point is determined upon each rate curve equal to 81.7% of the normal consumption, a percentage curve, Fig. 12, is secured which, when expanded by the $S^{0.54}$ ratio, passes through the point, $Q = 2.16$ mgd, $H = 116.5$ ft, and an interpolated 2.64-mgd rate curve—the same solution as previously determined.

At their intersection with the Y-axis, these curves also show the loss of head in the system with no flow at Point X. Abscissas to the left of the Y-axis, therefore, represent "put-ins" which are required at Point X to satisfy the requirements of draft in the system. Furthermore, with the extension of these curves to the X-axis the abscissas of the intersections would represent the quantities that must be introduced at Point X to satisfy the normal drafts

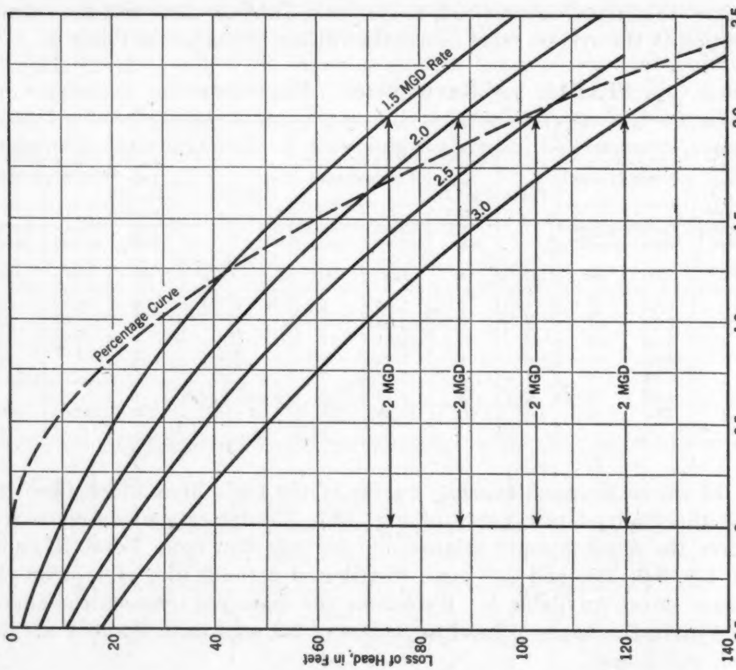


FIG. 12.—TYPICAL CASE I; LOSS OF HEAD IN LINE A X; VARYING DAILY DRAFT

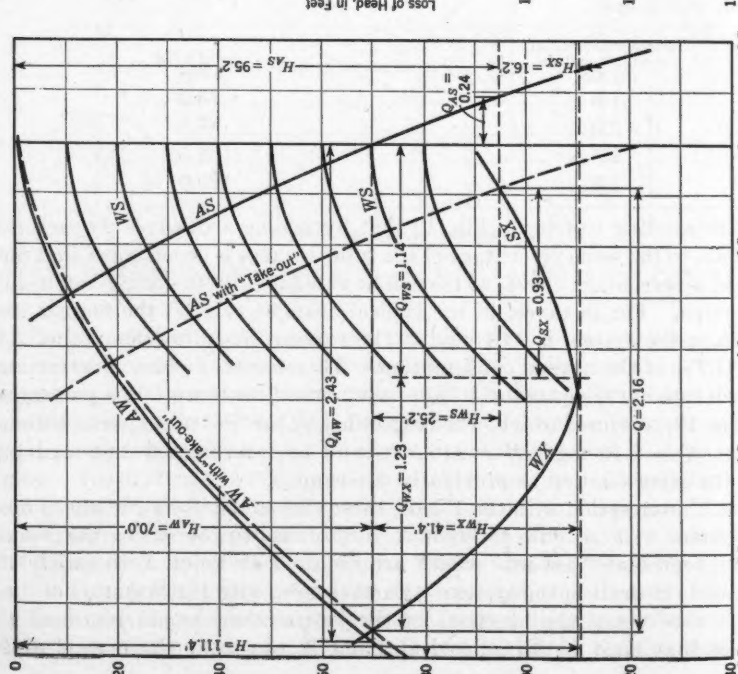


FIG. 11.—TYPICAL CASE I; TOTAL LOSS OF HEAD; RATE, 2.5 MILLION GALLONS DAILY; VARYING DAILY DRAFT

and maintain Points A and X at the same hydraulic grade; that is, with no total loss of head between the two points.

From Fig. 12, it is possible to study the effect of storage at, or coming into, the system at Point X. For instance, if a curve is drawn on the left of the Y-axis to represent the loss of head in a main bringing water from storage, the hydraulic grades at Point X and at storage, and the flows from each source, can be obtained directly under varying rates of draft in the system. If the storage is variable in elevation, this curve will move, as required, up or down the Y-axis during the time interval being studied. In this manner the size and elevation of storage best suited for the requirements can be determined.

Having determined the losses of head in, and the quantities flowing through, the various elements, it is a simple matter to check back along the lines to determine the hydraulic grades at other points and to prepare a pressure contour map.

Conclusions

The graphical analysis of some of the common hydraulic problems arising in the design of the transmission and distribution elements of a water plant represents an approach to the solution of problems which, by their nature, contain so many variable factors as to cause most designers to hazard guesses upon the results which they hope to obtain, based upon premises which may be at wide variance with actual facts.

In spite of the great sums of money invested in these facilities, no element of a water system receives less attention in design than the transmission and distribution system. Proper consideration given to the correct interpretation of the data upon which the design of these elements must be based, and to a study of the hydraulics involved, in conjunction with field tests which are helpful in planning extensions or improvements to the system, will go far in providing adequate and efficient installations with a minimum outlay of capital.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS

AERATION TANKS FOR ACTIVATED SLUDGE PLANTS

By S. W. Freese,[1] M. Am. Soc. C. E.

Synopsis

The design of aeration tanks in which diffused air is used for aeration, as differentiated from tanks with mechanical, surface aeration apparatus, is treated in this paper. An endeavor has been made to present the bases of theory and practice governing the design of the principal elements of diffused-air aeration tanks. The derivation of practicable bases of design has been made possible by the wealth of research, experimental, and plant-operation data which have been made available during the past few years particularly through the work of such agencies as the Sanitary District of Chicago.

Introduction

The results of experiments and plant-operation data offer rational bases for the design of the elements of diffused-air aeration tanks for the activated sludge process. Experiments run in parallel, with one element varying as between two experimental units, and plants that are adjusted to optimum operating conditions, are particularly good design guides.

The required capacity of the tanks depends upon quantity of sewage, aeration period, and percentage of return sludge. The results at plants and experimental units treating sewage of widely varying strengths indicate that the minimum aeration period and optimum percentage of return sludge vary with the strength of the sewage being treated. Parallel experiments with aeration tanks, 10 ft and 15 ft deep, indicate that between these two depths the economic depth of tank may be determined by balancing the cost of compressing air against fixed charges on tank cost. Experiments at Chicago, Ill., show that a ratio of tank width to tank depth of 2 to 1 may be used safely.

Data from a number of plants indicate that, except possibly in the case of some unusual trade wastes, the quantity of air required is approximately 0.0045 cu ft per gal of sewage per ppm of 5-day bio-chemical oxygen demand

Note.—Presented at the meeting of the Sanitary Engineering Division, New York, N. Y., January 16, 1936. Discussion on this paper will be closed in **February, 1938,** *Proceedings*.

[1] Cons. Engr. (Hawley, Freese & Nichols), Fort Worth, Tex.

(B.O.D.) removed over-all by plants using air-diffusion aeration tanks of the spiral-flow type. The "spiral-flow" type of tank requires less air than the "ridge and furrow" type, and the combination of air diffusion with mechanical agitation of the paddle-wheel type offers a further possibility of material savings in both air and total power required. The data from a number of plants indicate that the quantity of air required for "complete treatment" is independent of the degree of preliminary sedimentation and that plants with no preliminary sedimentation require no more air, in parts per million of over-all bio-chemical oxygen demand removal, than plants with preliminary sedimentation.

In the matter of tank details the use of deflectors placed longitudinally at corners of the aeration channels is now standard practice. Recent experiences indicate that diffusers with permeabilities of from 35 to 45 may be used safely, and that the size of the air bubbles does not differ appreciably from those diffused from plates of lesser permeability. The plates of higher permeability do not clog as readily, and they offer less resistance to the flow of air than plates of lower permeability (15±) which have been used in the past. Cement grout for jointing diffusion plates into plate containers is still common practice. The development of corrugated rubber gaskets with clips for holding the plates in place offers a possibility in overcoming the objection to cement grout joints which prevent the removal of plates intact.

Historical

In his classic "Modern Methods of Sewage Disposal," published in 1894, Mr. George E. Waring, Jr., devoted a chapter to "Filtration with Aeration," in which he reviewed the work done prior to 1894 in the matter of the aeration of sewage. Mr. Waring concluded the chapter with the following prophetic statements:

> "Probably if a mass of sewage could be held in a state of continual effervescence, due to injected air, for a sufficiently long time, its organic matter would be as completely oxidized as it would be in the best-regulated filtration; as it is, indeed, in a rapid river, after a certain period of flow. It is also conceivable that a deep and narrow channel, with an air-supply at its bottom, would purify a slow stream flowing through it for a practicable distance. It seems likely, however, that air introduced under pressure would be much more economically used in the pores of a coarse filter than in a freely moving body of liquid, through which it would rise in bubbles so rapidly as to come less completely in contact with the suspended and dissolved matters.
>
> "This whole subject is still in a speculative stage, and it is introduced here chiefly with a view to suggesting further investigation in an interesting and promising field."

The later development of the activated sludge process of sewage treatment and more recent experiences in the design of aeration tanks have confirmed the wisdom of these early statements.

Early experiments with the aeration of sewage prior to the development of the activated sludge process of sewage treatment may be conveniently reviewed in the first chapter of "The Activated Sludge Process," (1927), by Mr. A. J. Martin. In 1914 and 1915 Messrs. Edward Ardern and William T.

Lockett described, in a series of three papers read before the Manchester (England) Section of the Society of Chemical Industry, their experiments which are generally credited with having originated the activated sludge process of sewage treatment. Messrs. Ardern and Lockett gave credit to Mr. C. J. Fowler for his suggestions which led to, and were followed in, the making of the activated sludge experiments.

The process was patented by Jones and Attwood, Limited, Stourbridge, England, in a series of patents taken out in 1914 and 1915 in England and later perfected in the United States. Messrs. Jones and Attwood designed the first activated sludge plant of any size (approximately 900 000 U. S. gal per day) in 1916, at Worcester, England, part of an existing sedimentation tank being converted into aeration and sludge settling tanks. The aeration tank consisted of a battery of five channels, 8 ft wide by 80 ft long, 17 to 18 ft deep (water depth), and with three vertical under-flow baffles in each channel. Air diffusion was of the transverse ridge and furrow type with rows of diffusers at 5-ft centers in the first channel and at 10-ft centers in the remaining channels. The ratio of air diffuser area to tank area was approximately 1 to 10. The sewage passed through the channels in a modified "endless channel" type of continuous flow, at the end of each channel a portion of the flow being carried back by induced circulation into the preceding channel.

Laboratory experiments at Manchester were followed by the installation of an activated sludge sewage treatment plant at the Withington Sewage Works of the Manchester Corporation, this plant being placed in operation in September, 1917. The Withington aeration tank was constructed by converting a part of an existing sedimentation tank, approximately 20 ft by 100 ft, into a series of five channels, 4 ft wide and 100 ft long, with 5.5-ft water depth over diffusers. The tank was of the series, "once through," continuous flow type. The floor was laid out in transverse ridges and furrows, the slopes of the ridges being at 35° with the horizontal, and the air being introduced through rows of diffusers placed in the furrows, on 6.75-ft centers. The diffuser area amounted to one-seventh of the aeration tank area or the water surface area. This plant successfully treated 440 000 gal (U. S.) of domestic sewage per day. At another Manchester Corporation plant, the Davyhulme Sewage Works, the "spiral-flow" type of aeration tank was first used, the plant being put in operation in 1921. The aeration channels at Davyhulme are 8 ft wide by 9 ft deep (water depth) with diffusers along one side of the channel at the base of the curved footings of division walls; the ratio of diffuser plates to tank area is approximately 1 to 18. The plant has a rated capacity of 1 200 000 gal (U. S.) per day.

During 1915 a number of experiments with the activated sludge process were initiated in the United States, notably at Champaign, Ill., Lawrence, Mass., Baltimore, Md., Milwaukee, Wis., Brooklyn, N. Y., Chicago, Ill., Houston, Tex., and Cleveland, Ohio.[2] Based upon the experimental work at Houston and Milwaukee, Mr. E. E. Sands designed two large activated sludge plants for Houston. The North Side Plant at Houston, with a nominal capacity of 10 000 000 gal per day, was placed in operation in May, 1917, and the

[2] *Proceedings*, 1916 Convention, Am. Soc. of Municipal Improvements.

South Side Plant, with a nominal capacity of 5 000 000 gal per day, was placed in operation in August, 1918. These two plants were the largest activated sludge plants in operation until 1925 when the Milwaukee and Indianapolis plants were placed in operation.

The Houston aeration tanks consist of four units at the North Side Plant and two units at the South Side Plant. Each aeration unit consists of two channels, one 18 ft wide and the other 9 ft wide, of a length of 280 ft and a water depth over diffuser plates of 9.75 ft. The bottoms of tanks are of the sawtooth ridge and furrow type with rows of plates at 5-ft centers, the individual plates being spaced so as to give a ratio of diffuser-plate area to tank area of 1 to 7. For several years the plants were operated as the continuous flow, "endless channel" type, with a ratio of incoming raw sewage to circulating mixed liquor of from one-third to two-thirds; however, the North Side Plant has since been enlarged and remodeled as a "once through" type with provision for the re-aeration of return sludge.

The successful operation of the pioneer activated sludge sewage treatment plants, such as those at Manchester, Houston, and Worcester, along with the extensive experimental work at Chicago, Milwaukee, and Indianapolis, and elsewhere, not only firmly established the activated sludge process, but also gave a fairly rational basis for the design of aeration tanks and the other principal features of activated sludge plants.

Aeration Tank Capacity

The first step in the design of aeration tanks is to determine the required capacity. This capacity depends upon three factors: (1) The quantity of sewage to be treated; (2) the percentage of return activated sludge; and (3) the

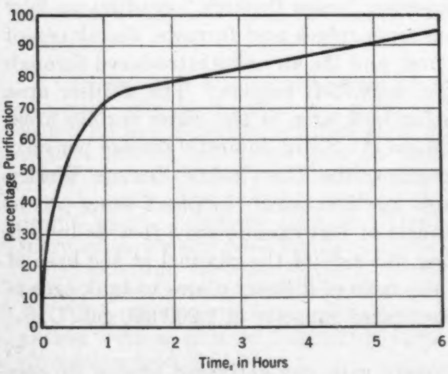

Fig. 1.—Relationship Between Aeration Period and Degree of Purification

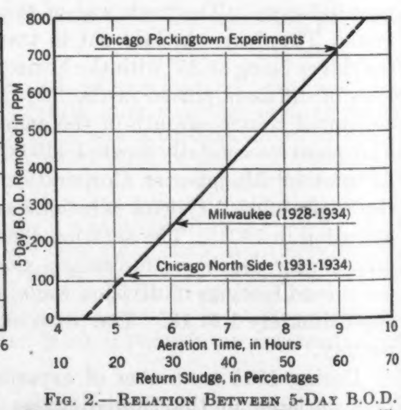

Fig. 2.—Relation Between 5-Day B.O.D. Removal and (1) Aeration Period, or (2) Percentage of Return Sludge

aeration period or time of detention of the mixed sewage and activated sludge liquor in the aeration tanks. Design data for aeration tanks have necessarily been related to the average daily sewage flow, and the writer has used that basis in considering the various factors involved. The average daily sewage

flow is a practical basis of design for works treating the average domestic-industrial sewage collected in separate sewer systems. The ratio of maximum hourly flows to daily averages may vary considerably without disturbing the activated sludge process, due to the capability of the sludge to store "biological energy."

Other factors being constant, the length of the aeration period depends primarily upon the quality of the sewage to be treated and the degree of treatment required. Fig. 1 is a typical curve showing the relationship between the aeration period and the degree of purification, the latter being measured after clarification of the mixed liquor. For a particular sewage this curve will vary with the percentage of return activated sludge, mixed with the sewage, the character and biological activity of the sludge, the kind and intensity of aeration, the time and type of clarification, temperature, and other factors of less importance.

During the first stage of the aeration period the impurities in the sewage are rapidly coagulated and adsorbed by the activated sludge, and the rate of coagulation adsorption during this first stage is proportional to the percentage of biologically active sludge in the mixed liquor.[3] After the initial coagulation adsorption has taken place, the purification of sewage, measured in terms of bio-chemical oxygen demand, proceeds at the much slower and fairly uniform rate of oxygenation of the mixed liquor. During this second stage the presence of a large proportion of sludge probably retards the oxygenation of the sewage proper because of the relatively greater oxygen requirements of the sludge.[4] The oxygenation of the mixed liquor during both stages, and of the sewage during the second stage, no doubt proceeds in accordance with the law expressed by E. B. Phelps: "The rate of biochemical oxidation of organic matter is proportional to the remaining concentration of unoxidized substance, measured in terms of oxidizability." [5]

For "complete treatment" the minimum aeration period varies with the strength of the sewage. To show this relationship the writer has plotted on Fig. 2 the aeration period against the 5-day B.O.D. removed for sewages of widely differing strengths—that is, the weak North Side Chicago sewage, the comparatively strong Milwaukee sewage, and the very strong Chicago Packingtown sewage. Both the Chicago North Side and the Milwaukee plants were loaded to capacity and were operated so as to obtain the optimum results during the years included in the study. The Chicago Packingtown activated sludge experiments were conducted with the idea of obtaining optimum plant operation characteristics. In each case a high degree of purification was accomplished. The sludge is not re-aerated at Chicago and Milwaukee, and the data used from the Chicago Packingtown experiments cover only those in which the sludge was not re-aerated.

[3] "Observations on Biological and Physical Properties of Activated Sludge and the Principles of Its Application," by F. W. Harris, T. Cockburn, and T. Anderson, *Proceedings*, The Assoc. of Managers of Sewage Disposal Works, 1926, p. 52 *et seq.*

[4] "The Oxygen Requirements of the Activated Sludge Process," by S. Grant, E. Hurwitz, and F. W. Mohlman, *Sewage Works Journal*, Vol. II, No. 2 (April, 1930), p. 228 *et seq.*

[5] "The Disinfection of Sewage and Sewage Filter Effluents," by E. B. Phelps, *Water Supply Paper 229*, U. S. Geological Survey, 1909, pp. 74–78.

The aeration-period curve shown in Fig. 2 is plotted from the data given in Table 1(*a*). In Fig. 2 the point plotted for the Chicago North Side Plant is a practicable minimum of aeration period for a weak sewage. The aeration period plotted for Milwaukee (6.3 hr) is the average total aeration time, of

TABLE 1.—CHARACTERISTICS OF "COMPLETE TREATMENT" OF SEWAGES OF DIFFERENT STRENGTHS

Plant	Period	(*a*) AERATION PERIODS REQUIRED					(*b*) OPTIMUM PERCENTAGES OF RETURN SLUDGE		
		Five-Day Bio-Chemical Oxygen Demand							
		Raw sewage, in parts per million	Effluent, in parts per million	Plant removal, in parts per million	Plant removal (percentages)	Aeration period, in hours	Return sludge (percentage of incoming sewage)	Solids in return sludge (percentage)	Volatile matter in sludge solids (percentage)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
Chicago, Ill., North Side	1931–1934	115.8	11.3	104.5	90.3	5.2	21.7	1.39	67.7
Milwaukee, Wis.	1928–1934	270.1	13.1	257.0	95.2	6.3	32.5	1.19	76.6¶
Chicago, Ill., Packingtown experiment*	1916–1917†	752.0‡	32.0§	720.0	96.0	9.0	60.0	0.85‖	62.6**

* "Report on Industrial Wastes from the Stockyards and Packingtown in Chicago" (Vol. II, 1921); The Sanitary District of Chicago. † 15 runs. ‡ 10-day B.O.D., 990 ppm. § 10-day B.O.D., 42 ppm. ‖ Minimum, 0.4, and maximum, 1.3. ¶ Average of 1933 and 1934. ** Minimum, 55, and maximum, 73.

which 5.44 hr constitutes the time in the aeration tanks proper; however, a material part of the difference is accounted for by the time of detention of the mixed liquor in the mixing channels. In the large scale experiments conducted by the Sanitary District of Chicago with Packingtown sewage in 1916 and 1917, a period of 9 hr was found to be a practicable minimum aeration time without re-aeration of the sludge.

The second important determination to be made in arriving at the aeration tank capacity is the percentage of return activated sludge. In the foregoing discussion of minimum aeration periods and, particularly in arriving at the aeration period curve shown in Fig. 2, it was assumed that the optimum percentage of sludge was being returned. The return of a large percentage of biologically active sludge will provide a quick coagulation adsorption of the impurities in the sewage, the time being inversely proportional to the percentage of sludge. However, after the initial coagulation adsorption has taken place, the larger percentages of return sludge seem to "rob" the sewage as the further oxygenation proceeds. Too small a percentage of return sludge (that is, less than that required to maintain biological activity) will require re-aeration of the sludge.

Without re-aeration of the sludge it appears, therefore, that the theoretical optimum percentage of return sludge is that percentage which will permit the required purification in a minimum of time and, at the same time, maintain an equilibrium of biological activity in the sludge. In general, the minimum

percentage of biologically active return sludge will mean the minimum time for "complete" purification of the sewage. On the other hand, if this percentage of return sludge drops too low, the sewage will not remain biologically active.

As was the case with the aeration period and in the absence of experiments with the particular sewage, the best guide as to the optimum percentage of return sludge is the experience of plant operators, or of experimentation in plants where the percentage giving the best results has been determined.

As would be expected from theoretical considerations, the optimum percentage of return sludge found in the practical operation of activated sludge plants varies with the strength of the sewage in much the same manner as the aeration period. The optimum percentage of return sludge has been plotted against the 5-day B.O.D. removed for the same three sewages of widely differing strengths for which the aeration periods were plotted against 5-day B.O.D. removed. The data are such that the two curves may be plotted as an identical curve, as has been done in Fig. 2. The curve showing percentage of return sludge, in Fig. 2, is plotted from data in Table 1(*b*).

Although the curve for percentage of return sludge, shown in Fig. 2, does not take into consideration the percentage of solids in the activated return sludge, the determination of the optimum percentage of return sludge is usually related to the parts per million of solids maintained in the mixed sewage and activated sludge liquor.

In referring to the operation of the North Side Plant, in Chicago, treating a weak sewage with a 5-day B.O.D. of 116 ppm, Langdon Pearse, M. Am. Soc. C. E., states that it has been found practical to operate the plant with a smaller proportion of solids in the mixed liquor than the 3 000 ppm formerly assumed, and that by operating with approximately 2 000 ppm more flexibility is secured, with opportunity to store the increase of solids produced by storm flows. Mr. Pearse also found that some saving in air resulted and that it was practicable to cut the percentage of return sludge in volume, below 20.[6]

At the Milwaukee Plant with a sewage having a 5-day B.O.D. of 270 ppm, D. W. Townsend, M. Am. Soc. C. E., reports that it has been the custom in operating this plant to maintain a mixed liquor suspended solids content of approximately 3 000 ppm (0.3 of 1%), this solids content within reasonable working limits having been found to be about optimum for the general conditions encountered.[7]

The percentage of volatile matter in the sludge also affects the characteristics of the activated return sludge at least in so far as the rate of absorption of oxygen by the sludge is concerned.[4] However, few data are available to the writer as to how this affects the optimum percentage of return sludge.

The foregoing comments have been confined to the minimum practicable aeration period and optimum sludge return required for substantially complete treatment, as differentiated from partial treatment, and to the minimum time

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[6] "Comparison of Original Design Basis for North Side Treatment Works with Present Accepted Practice," by Langdon Pearse, Report to Public Works Administration, Board of Review, February 19, 1934.

[7] "Five Years Operation of Milwaukee Treatment Plant," by Darwin W. Townsend, *Proceedings*, Am. Soc. of Municipal Improvements, Vol. 39 (1933–1934), p. 189.

and optimum sludge return required without re-aeration of the sludge. Partial treatment and re-aeration of the sludge "go hand in hand." Re-aeration or re-activation of the sludge is required when the aeration period of the mixed liquor is insufficient to maintain either the biological activity or the oxygen requirements of the sludge, or both. With partial treatment, however, the demands on the sludge are so much greater and the required aeration period so much smaller that re-aeration of the sludge is required. Re-aeration is also required with either complete or partial treatment if the aeration period is kept at a minimum and the percentage of return sludge is too small, re-aeration being required because of the relatively heavy demands made on the sludge.

Even with complete treatment of the sewage, re-aeration of the sludge may be economical in many cases, particularly in the case of very strong sewagés or industrial wastes.

The Chicago Packingtown sewage required 9 hr of aeration without re-aeration of the sludge. With re-aeration the average aeration period in the mixed liquor tanks was 5.3 hr with 3.8 hr aeration in the re-aeration tanks, the weighted average being equivalent to a 6.6-hr period as compared to the 9-hr period without re-aeration. The settling period however had to be increased from 1.3 hr without re-aeration to 1.7 hr with re-aeration. The re-aeration experiments indicated a net saving of 20% of tank volume (aeration tanks and settling tanks).

Evidence as to the economy in tank volume required in the treatment of sewage of average strength, because of re-aeration of the sludge, is not so clear. Unless it is possible to conduct large-scale experiments with the particular sewage being studied, the safe procedure is to design aeration tanks on the basis of the minimum volume required without re-aeration, in line with the curve shown in Fig. 2, but to provide sufficient flexibility in the number of aeration tanks or channels such that varying periods of re-aeration may be tried. Return sludge connections can usually be provided so as to use several of the channels as re-aeration channels at a relatively small additional cost.

Depth of Tank

The use of a depth of 15 ft or, in some cases, a few inches less, for aeration tanks is almost universal in the large activated sludge plants in the United States. A number of diffused air plants in Great Britain use depths from 5 ft to 10 ft; however, the recently built and comparatively large plant at Manchester (16 mgd) uses a 15-ft depth. The plants at Houston have a depth over the diffuser plates of 9.75 ft. There are also a number of smaller plants in the United States using diffuser tubes at relatively shallow depths (5 ft, and less), but with tanks as deep as 15 ft.

In England the use of depths shallower than 10 ft can be largely attributed to the fact that many of the activated sludge plants were converted from existing works of other types of treatment where the existing tanks were of the shallower depths. Where the existing tanks were deeper, as at Worcester, England (17 to 18 ft), and at Reading, England (23.5 ft), the remodeled aeration tanks are of the same depth.

The prevalence of the 15-ft depth in the United States can be attributed largely to experience at Milwaukee and Chicago, where large-scale experiments were conducted with aeration tanks 10 ft deep, and 15 ft deep, run in parallel. The 15-ft depth was adopted at both Milwaukee and Chicago for the large activated sludge plants. At Milwaukee, the tests of the tanks 10 ft and 15 ft deep were discussed by the late Harrison P. Eddy, Past-President, Am. Soc. C. E., as follows:[8]

"The comparative efficiency of treatment in tanks 10 and 15 ft. in depth, was made the subject of careful and prolonged study at the Milwaukee Testing Station. Tests carried out with the greatest care, under the personal direction of William R. Copeland, Affiliate, Am. Soc. C. E., Chief Chemist, showed that, taking all the facts into consideration, it appeared that, for the same quantity of free air per gallon of sewage treated, there was little difference in the work accomplished by the 10-ft and the 15-ft tanks. The 15-ft tanks have produced an effluent of somewhat greater stability, whereas the 10-ft tanks have accomplished a slightly greater reduction in suspended matter and bacteria. If one depth of tank has an advantage over the other in purification efficiency, it seems to be the 15-ft. depth.

"An important consideration in the selection of depth of tank is the power required for compressing the air sufficiently to overcome the hydrostatic pressure of the several depths of sewage under consideration. In the cases of the 10 and 15-ft. depths, the latter would require 50% more power than the former, disregarding the effect of friction in pipe lines and porous plates. This saving will generally be offset, partly at least, by the increased cost of the larger area of tanks of the lesser depth. At Milwaukee, this is important because of the cost of the bulkhead, the sheet steel enclosure, and the pile foundation. Another important factor there is the limited area available for the treatment plant, as it is felt that additional area could not be devoted to this purpose, on account of the requirements for harbor development."

At Chicago, the sewage treatment works on the Des Plaines River were designed for experimental purposes with three aeration tanks of a depth of 10 ft and one of a depth of 15 ft. The results of the experiments as between the two depths, using the same volume of air per gallon of sewage, were summed up by Mr. S. L. Tolman, of the Sanitary District of Chicago, as follows:[9] "The operating data do not indicate any great difference in purification between these two depths."

Aside from the economy of construction cost of deep tanks over shallow tanks, another consideration is as to whether or not any appreciable oxygenation is accomplished by the air bubbles as they rise through the greater depth of liquor in the deeper tanks. As indicated by the Milwaukee and Chicago experiments this additional oxygenation in the deeper tanks is not material. This is probably due to two reasons: First, the bubbles rising through the liquid are enveloped in a surface tension film, including some colloidal matter (which film interferes with the passage of oxygen into the liquid at least until the bubble breaks the surface of the liquid[10]); and, second, the oxygen requirement of the sewage amounts to only a small percentage (not more than 10) of the oxygen in the air required for mixing and for maintaining aerobic conditions.

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[8] *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 864.

[9] "The Operation of the Des Plaines River Sewage Treatment Works and Small Plants of the Sanitary District of Chicago," *Journal*, Western Soc. of Engrs., Vol. XXXI, No. 7 (July, 1926).

[10] *Proceedings*, Royal Dublin Soc., 1920; *Journal*, Royal Sanitary Inst., October, 1926; and *Minutes of Proceedings*, Inst. C. E., Vol. CCXVII, pp. 136, 137, and 191.

In a series of experiments conducted by the writer in 1925, nitrogen and air (also oxygen and carbon dioxide) were used in parallel for agitation of the mixed liquor, the raw sewage, the percentage and quality of activated sludge, the volume of gas, the aeration period, the depth of liquor and the clarification period being identical for each gas. The air and the inert gas—nitrogen—gave practically the same results as may be judged from Table 2, which gives the results of the experiments.[11] The results of these experiments indicate that a large part of the oxygenation occurs as the bubbles break the surface of the mixed liquor.

TABLE 2.—COMPARISON OF NITROGEN AND AIR BUBBLE AGITATION

Description	Five-Day Bio-Chemical Oxygen Demand in Parts per Million		
	Medium strength sewage, Run No. 5	Strong sewage, Run No. 8	Average, Runs Nos. 1 to 8
Raw sewage	180	600	365
Effluent, using air agitation	16	40	24
Effluent, using nitrogen agitation	20	36	24

The foregoing considerations, particularly the results of the experiments at Milwaukee and Chicago, indicate that within practicable limits the economic depth of tank can be determined by balancing the greater cost of compressing air for the deeper tank against the lesser fixed charges for the aeration tanks of the greater depth. This problem may be readily analyzed by plotting the various costs against the corresponding depths.

The economic limit of depth is probably 15 ft to 16 ft, except in very extreme cases where the cost of site or of foundations (or of both) is very high. At Milwaukee, for example, where a 15-ft depth was used, the aeration tanks were built on "made" land and required bearing piles to support them. A number of British plants operate quite economically with depths of tank from 5 ft to 10 ft. Factors that would tend to increase the depth are, as follows: (*a*) Comparatively weak sewage requiring a small amount of air; (*b*) cheap power; (*c*) high construction costs; and (*d*) costly land for site. The City of Chicago is a typical example of a case in which these factors prevail. As a corollary, factors that would tend to decrease the depth are, as follows: (1) Comparatively strong sewage; (2) costly power; (3) low construction costs; and (4) inexpensive land.

In an attempt to gain the advantages of both the deep tank and the shallow tank a number of plants have been built in which the air diffusers, usually diffusion tubes, were placed above the mid-depth point in the tank, in some cases quite near the surface. In an aeration tank of this type it is essential that sufficient motion be imparted to the liquor to mix the sludge with the sewage and to prevent deposition of solids, as well as to maintain aerobic conditions in the mixed liquor.

[11] For a description of the experiments and analyses of oxygen consumed, dissolved oxygen, free ammonia, nitrates, and nitrites, as well as data covering other runs, see *Proceedings*, Texas Section, Am. Soc. C. E., Fall Meeting, 1925, pp. 13–36.

Width of Tank

Until quite recently, the ratio of width to depth of aeration tanks or channels has been maintained at less than 1.5, the British plants maintaining a ratio of approximately 1.0 and less. This was true of both the ridge and furrow and of the circulating or spiral-flow types of tanks. Inasmuch as the circulating or spiral-flow type of tank has largely superseded the ridge and furrow type due to the lesser air requirements of the circulating or spiral-flow type (as will be mentioned subsequently), this paper is confined to the latter type.

The new Cleveland Easterly Plant has a ratio of 1.8 (27-ft width by 15-ft water depth)[12] and the new Chicago Southwest Plant has a ratio of 2.1 (32.75-ft width by 15.5-ft water depth).[13]

The most careful observations available to the writer as to how wide the tank can be and still maintain good mixing and aeration are those made by the Sanitary District of Chicago at the North Side Plant. The observations were reported[6] as follows:

"In the plant design, the batteries were divided up into 12 water-tight tanks, each of which was divided by a longitudinal baffle into 2 channels, 420 feet long and 16 ft. 1¾ inches average width. These channels were equipped with 2 rows of diffuser plates, running longitudinally. However, in the westerly tank, the dividing wall was omitted, giving a separate tank 33 feet 3½ inches average width. Experiments were carried out with channels 25 feet and 33 feet 3½ inches width to determine the efficiency of aeration with three longitudinal rows of diffuser plates.

"Tests have shown that the effluent was equally as good with these widths as with the 16 ft. 1¾ inch width tank and no more air was required per gallon of sewage treated."

Based upon the operating data of the North Side Plant (0.4 cu ft of air per gal, 25% return sludge, and 4-hr aeration period), the use of air per foot of length for this particular channel may be assumed as between 4 and 5 cu ft per min. Inasmuch as the use of air at the North Side Chicago Plant, per gallon of mixed liquor per unit of time, is comparatively low, the foregoing observations indicate that a ratio of width to depth of as much as 2 is safe for a spiral-flow tank with diffusion plates along one side, unless the sewage is so weak as to require even less air than that at the Chicago North Side Plant.

At the treatment plant in San Antonio, Tex., the dividing wall was omitted between two aeration channels in order to observe the aeration in the wider channel. This channel has a ratio of width (40 ft 10 in.) to water depth over diffusion plates (14 ft 7.75 in.) of 2.8. The channel has a length of 150 ft, with three rows of diffuser plates longitudinally along each side of the channel and with an inverted V-shaped baffle, 3 ft high, longitudinally down the center of the channel. Fig. 3 shows a cross-section of the wide channel at San Antonio.

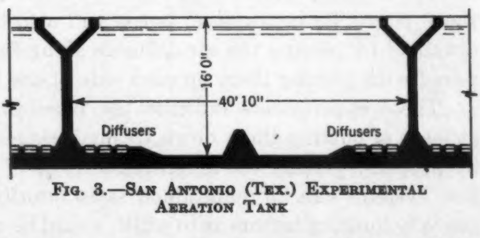


Fig. 3.—San Antonio (Tex.) Experimental Aeration Tank

[12] *Engineering News-Record*, Vol. 107, p. 1035.

[13] From Contract Plans, Southwest Sewage Treatment Works, The Sanitary District of Chicago.

During 1934 the use of air per foot of length of the channel averaged 8.5 cu ft per min, or 4.25 cu ft per min per ft of length of diffusion plate system at each side of the channel (three longitudinal rows of plates in each diffusion plate system).

The liquor surface in the wide experimental channel at San Antonio reveals no well-defined dividing line along the center of the tank between the two spiral flows. The spiral flows creep from one side to the other of the center line of the tank; this creeping varies constantly and it is not known whether this is helpful in the mixing-aeration process, or whether it tends to create dead spots. H. A. Hunter, Assoc. M. Am. Soc. C. E., and W. S. Stanley, Chief Chemist and Superintendent, Sewage Treatment Plant, San Antonio, Tex., have made tests recently with the wide experimental tank as compared with the 20-ft tanks with diffuser plates on one side only. The effluent from the wide unit showed a smaller 5-day B.O.D. (3.3 ppm, as compared with 9.3 ppm); however, the flow through the wide tank had been throttled down so much that there was a considerably longer detention time in the wide unit than in the 20-ft unit used in the comparison. Further tests will be required to determine whether or not the results obtained in the experimental double unit are the equal of those obtained in the single units.

In a letter dated January 6, 1936, L. C. Whittemore, M. Am. Soc. C. E., states that an experimental aeration tank similar to the one at San Antonio, except that it is 68 ft wide, has just been placed in operation at the Chicago Calumet Plant. As at San Antonio, the diffuser plates are placed along both sides of the channel. He states further:

"While it is, of course, too early to obtain any quantitative results or form any opinions as to the success of this arrangement, I saw the tank in operation the day sewage was put in it. The dividing line where the two currents from the sides of the tank was rather wavy and irregular, but there appeared to be good velocities maintained across the top of the tank toward the center."

Not only will the quantitative results at Calumet and more comparable results at San Antonio from the operation of the wide experimental units at these plants be instructive, but it will also be interesting to know the results obtained by placing the air diffusers along the center line of the tanks as compared with placing them on each side of the tank.

These experiments indicate the possibility of eliminating baffle-walls, or at least of cutting them down to small ridges midway between rows of plates, by properly placing the longitudinal rows of systems of plates. If the spiral-flow systems can be maintained satisfactorily by the energy of the air alone, the only limiting factors as to width would be a ratio of length to width sufficient to avoid short circuiting through the tank and a sufficient number of tanks to provide flexibility of operation.

Until such experimental works prove otherwise, existing data indicate a limit of the ratio of width to depth of approximately 2 to 1 for channels with diffusion plates on one side. The ratios should be modified to some extent depending upon the quantity of air necessary in the particular case.

Quantity of Air

The introduction of air in an air diffusion type of activated sludge plant serves two purposes: First, it mixes the sewage with the coagulating-adsorbing activated sludge and maintains the contact between the particles of activated sludge and the impurities in the sewage; and, second, it maintains aerobic conditions for the oxygenation of the mixed sewage-sludge liquor, or the sludge (or both), such that the sludge, or at least the return sludge, will be in a state of equilibrium in so far as its biological activity is concerned. The meeting of the second condition (the introduction of sufficient air to maintain the activity of the sludge) will more than satisfy the first condition as to the mixing of the sludge with the sewage, except possibly in the case of partial treatment plants requiring re-aeration of the sludge.

The air required to maintain aerobic conditions in the mixed liquor, or sludge (or both), for the required aeration period is approximately ten times that required to meet the bio-chemical oxygen demand of the sewage in an efficiently operated air-diffusion plant. Even then, a large part of the oxygen demand is no doubt met by oxygen from the atmosphere rather than from the diffused air introduced into the sewage, the atmospheric oxygenation taking place as air bubbles break the surface of the liquid. The quantity of air necessary to maintain sufficiently aerobic conditions for the oxygenation of the sewage or sludge (or both) is directly proportional, within a fairly narrow range, to the bio-chemical oxygen demand removed in an efficiently operated air-diffusion plant.

TABLE 3.—Air Required per Parts per Million of Five-Day Bio-Chemical Oxygen Demand Removed

Plant	Length of record, in months	Five-Day Bio-Chemical Oxygen Demand, in Parts per Million			Quantity of Air	
		Raw sewage	Plant effluent	Over-all plant removal	In cubic feet per gallon of sewage	In cubic feet per gallon per parts per million. 5-day B.O.D. removed
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Chicago North Side (26 min preliminary sedimentation)	18	112	9	103	0.46	0.0045
Chicago North Side (preliminary sedimentation for 5.4 months only)	42	116	13	103	0.39	0.0038
Milwaukee (fine screens)	84	270	17	253	1.15	0.0045
San Antonio (41-min preliminary sedimentation)	61	196	15	181	0.97	0.0054
Indianapolis (fine screens)	96	287	35	252	1.13	0.0045
Chicago Packingtown	15 runs	752	32	720	3.06	0.0042

Table 3 shows the quantity of air required for a number of plants per gallon of sewage per part per million of bio-chemical oxygen demand removed. Table 3 also shows the nature of the preliminary treatment. Referring to Column (6), Table 3, the quantity of air used by the Milwaukee "ridge and

furrow" type of aeration tanks (1.53 cu ft per gal) has been reduced by 25% (1.15) to obtain the comparison with the "spiral-flow" tanks used in the table. One-half the aeration tanks used in the Chicago Packingtown experiments were of the "ridge and furrow" type and air used in the experiments (3.5 cu ft per gal) has been reduced 12.5% to obtain (3.06) an approximation of the corresponding "spiral-flow" figure.

During the last twenty-four months of the Indianapolis record the plant was operated largely as a straight aeration plant rather than as an activated sludge plant. During this period (1933 and 1934) the plant reduction in parts per million of 5-day B.O.D. was from 256 to 94, making an over-all plant removal of 162 ppm, with a use of air of 0.64 cu ft per gal of sewage treated.

The quantity of free air required per part of bio-chemical oxygen demand removed over-all by a "complete treatment" air-diffusion plant seems to be largely unaffected by, and independent of, the aeration period, the percentage of return sludge, re-aeration of sludge, preliminary treatment by screens, or settling, or by depth of aeration tank. This assumes efficient plant operation and sufficient final clarification. The arrangement of the diffuser plates at the bottom of the aeration tanks, however, has a decided effect on the air required to maintain aerobic conditions in the mixed liquor. It has been demonstrated at a number of plants, particularly in experimental and full-scale units at Manchester, Chicago, and Indianapolis, that the "spiral-flow" type of tanks with the diffuser plates longitudinally along one side of the channels require much less air to accomplish the same results than do the "ridge and furrow" type of aeration tank with the rows of diffuser plates cross-wise of the channels. The air saved by the "spiral-flow" type of tank over the "ridge and furrow" type approximates 25 per cent.[9, 14]

It is to be noted from Table 3 that the omission of preliminary treatment does not adversely affect the quantity of air required per part per million of 5-day B.O.D. removed by the plant as a whole. This is due no doubt to the fact that any solids removed by the preliminary treatment would settle out in any case in the final clarifiers and to the further fact that those solids are of such a nature that they will absorb a negligible quantity only, of oxygen as they pass through the aeration tanks.

The fact that Table 3 shows such uniform results with respect to the quantity of air required per part per million of 5-day B.O.D. removed by the over-all plant, in spite of the fact that a large percentage (30 to 40% in some cases) of the 5-day B.O.D. had been removed in the preliminary settling tanks in some of the plants, is attributable to the fact that the ratio to 5-day B.O.D. of suspended solids (and 5-day B.O.D. associated with those solids) is approximately the same for each of the plants for which comparable data were available. It is questionable whether the air consumption per part per million of bio-chemical oxygen demand removed, indicated in Table 3, would hold good for a strong industrial waste with a relatively small volume of settleable solids.

It is desirable to remove the grit from the sewage so that it will not settle in the aeration tanks or require too great a quantity of air to keep it in circula-

[14] "Sewage Aeration by Diffused Air," by Frank C. Roe, *Sewage Works Journal*, Vol. V, No. 5 (September, 1933).

tion. It has also been considered advisable from the standpoint of sludge disposal to settle out a considerable part of the solids in the preliminary tanks so as to obtain the higher concentration of solids (approximately 5%, or more) obtainable in the preliminary tanks as compared with the concentration obtainable in the final clarifiers (1% ±). However, there is some question as to whether the over-all concentration of solids from both the preliminary and the final tanks is any greater with the use of preliminary settling tanks. In a report to the Board of Review, Public Works Administration, for the West-Southwest Plant, Mr. Pearse stated,[6] as follows, with reference to preliminary settling tanks:

"In the original design, a preliminary settling period of 30 minutes on 175 m.g.d. was thought advantageous to reduce the settling solids and cut the use of air, based on experience at Calumet. Operating results for 18 months have indicated, however, on the North Side Plant, that the preliminary tanks are not necessary from the sewage treatment standpoint, as there is practically no difference in the amount of air used. Further, there has been no need of their use as skimming tanks for floating material. However, in the dewatering of the sludge the mixture of some fresh sludge with the activated sludge appears to aid in obtaining a higher rate of solid cake per square foot from the vacuum filters."

From the data available, therefore, it seems safe in the design of aeration tanks for "complete treatment" plants (without paddle-wheels or other mechanical aids to aeration), treating sewage without any unusual trade wastes, to provide 0.005 cu ft of free air per gal of sewage (average flow) per ppm of 5-day B.O.D. to be removed from the raw sewage by the over-all plant. Sufficient flexibility should be provided in the number and capacity of air compressors to permit a reduction to 0.004 cu ft (or even lower) of free air, should the actual operation of the plant permit of this minimum usage of air.

Considered entirely as an agitation device, a modern air-diffusion type of plant has an efficiency from "wire to sewage" of between 60% and 70%, depending to some extent upon the size and type of air compressors used. In other words, the work done on the sewage after taking care of all motor, compressor, air-piping, and diffuser-plate losses, approximates 60% to 70% of the energy delivered to the motor in a modern plant. (See operating data for Chicago North Side Plant and San Antonio.) Although these efficiencies were computed for plants using electrical power, plants using turbine-driven compressors show approximately the same efficiencies for the corresponding operations.

However, the function of an aeration tank is not agitation, in itself, but the maintenance of contact between the activated sludge and the sewage, under aerobic conditions. One form of agitation with air diffusion may be much more effective than another form, as is shown by the improvement of the "spiral-flow" form of aeration over the "ridge and furrow" type. There is promise of further improvement by utilizing a combination of air-diffusion agitation with mechanical agitation, such as the combination being utilized by Karl Imhoff, M. Am. Soc. C. E., at the sewage treatment plant in Essen-Rellinghausen, Germany, in which a submerged paddle mechanism revolves on a horizontal shaft running the length of the aeration channel in a direction

opposite to the motion imparted by the air-diffuser plates located longitudinally along the side of the channel as shown in Fig. 4. Dr. Imhoff reports that with diffused air from 0.8 to 1.12 cu ft per gal and 24 hp per mgd were needed, but that with paddles in operation 0.16 cu ft of air per gal and 8.4 hp per mgd were found to suffice.[15] Mr. F. W. Mohlman and Charles E. Wheeler,

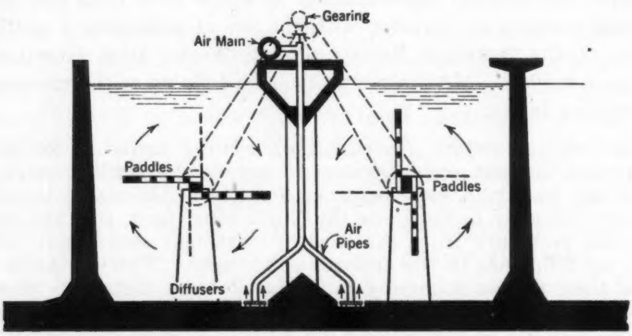


FIG. 4.—CROSS-SECTION OF AERATION TANK EQUIPPED WITH AIR DIFFUSERS AND PADDLE-WHEELS

Jr., Assoc. M. Am. Soc. C. E., of the Sanitary District of Chicago, reported similar results obtained on large-scale experiments (1 500 000 gal per day) with combination air-diffusion and paddle-wheel agitation. They found that it would require 0.80 cu ft of air per gal, with air alone, and a 5-hr aeration period to produce an effluent comparable with that produced by the paddle-wheel plus air, with 0.20 cu ft of air per gal of sewage and a 5-hr period. The paddles required 6.7 hp and the air 6.4 hp, giving a total of 13.1 hp per million gallons. With air alone at 0.8 cu ft per gal, and a 5-hr period, the total requirement would be 26 hp per million gallons.[16]

AERATION TANK DETAILS

General Layout.—The laying out of the aeration tanks is entirely a matter of the most economical arrangement for the local conditions encountered. Plants of the air-diffusion type can be divided in general into two classes: (1) Those with an even number of channels in each aeration tank battery, arranged so that the inlet and outlet are at the same end of each tank battery of channels; and (2) those with an odd number of channels in each battery, arranged so that the inlet and outlet are at opposite ends of each battery. Conditions, any one of which may favor Class (2), are: (*a*) Incoming sewage main entering one side of plant site, with the effluent outfall out of plant site necessarily on opposite side; (*b*) re-aeration of return sludge, thus allowing return to the opposite side of the aeration tank battery through aeration channels; and (*c*) sludge return by pumping.

Conditions, any one of which may favor Class (1), are: (*d*) Plant size requiring "pipe gallery"; (*e*) incoming sewage main and effluent outfall

[15] "The Activated Sludge Process," by A. J. Martin, Macdonald and Evans, Lond., 1927, p. 252.

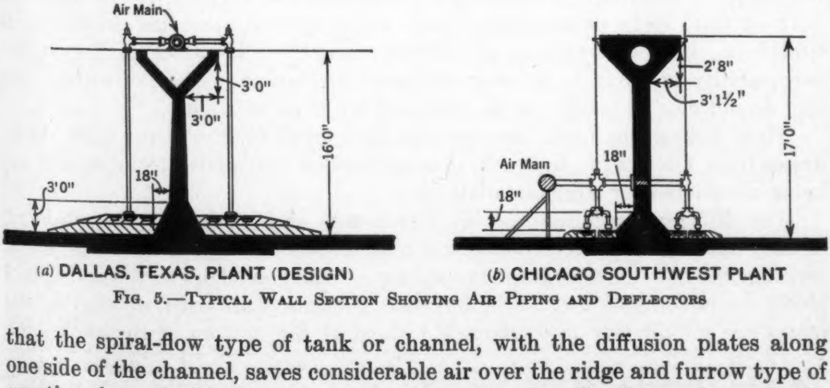
[16] "Activated Sludge Experiments at the Calumet Sewage Treatment Works," by F. W. Mohlman and C. E. Wheeler, Jr., Assoc. M. Am. Soc. C. E., *Sewage Works Journal*, Vol. II, No. 4 (October, 1930).

entering and leaving the side of the plant site; and (*f*) sludge return by air lift.

The aeration tanks should be divided into a sufficient number of batteries with pressure-bearing division walls between batteries to provide flexibility of operation and also to provide for the repair of diffuser plates, plate containers, piping, etc.; they should also be arranged so that the longitudinal velocity through the channels is low enough to avoid any undue loss of head. Otherwise, the number of batteries of aeration channels should be kept at a minimum in order to reduce the total number and cost of inlets and outlets. Baffle-walls between channels are usually non-pressure bearing, and division walls between batteries may be non-pressure bearing if there is a sufficient number of groups of batteries with pressure-bearing division walls between each group to provide flexibility of operation. In case division walls are made non-pressure bearing, a "fool-proof" arrangement should be provided for the draining of the batteries on the two sides of the non-pressure-bearing division wall, at equal rates. The openings for the passage of mixed liquor from one channel to another in the same battery should be large enough to avoid any appreciable head losses and should be at the bottom of the baffle-wall. The openings between channels in the new Southwest Sewage Treatment Works, at Chicago, are 10 ft high by 8 ft wide.

As previously stated, provision for reserving a part of the channels as sludge re-activation tanks can usually be made at a relatively small additional cost. This seems advisable in the case of strong sewages particularly in the absence of experimental plant data with the particular sewage to be treated. Aeration channels for mixing the return sludge with the sewage, influent channels into the different batteries, and effluent channels from the different batteries can often be made of substantially the same dimensions as the channels in the aeration tank batteries proper, usually at some saving in over-all plant cost and at no sacrifice in operating costs.

Section of Channels.—The capacity of tanks and the depth and width of channels have already been discussed. Since the development of the fact



(*a*) DALLAS, TEXAS, PLANT (DESIGN) (*b*) CHICAGO SOUTHWEST PLANT

FIG. 5.—TYPICAL WALL SECTION SHOWING AIR PIPING AND DEFLECTORS

that the spiral-flow type of tank or channel, with the diffusion plates along one side of the channel, saves considerable air over the ridge and furrow type of aeration channel with plates across the channel (see heading "Quantity of Air"), practically all new plants have utilized the spiral-flow type of tank. The main

object to be attained in this type is to maintain the spiral flow with a minimum of dead spaces, eddying, and friction losses. As a result of considerable experimentation corner deflectors at an angle of approximately 45° with the vertical are generally used. Fig. 5 shows two typical examples of channel cross-sections with the dimensions of corner deflectors.

Diffuser Plates.—Tank dimensions, aeration period, percentage of return sludge, and quantity of air per gallon of sewage having been determined, the area of diffuser plates required is, within certain limits, a matter of economic balance between the cost of diffuser-plate installation and the cost of compressing air to take care of the pressure loss through the plates, the greater the number of plates the less the pressure loss. The practicable range of air quantity per square foot of plate seems to lie between 1 and 4 cu ft per min, operating difficulties being encountered outside these limits. As indicated by the experience of the Chicago North Side and San Antonio Plants, 4 to 6 cu ft of air per min per ft of channel will induce a circulatory flow motion in channels 15 ft deep and 33 to 40 ft wide, sufficient to prevent deposition of solids from grit-free mixed liquor (see heading "Aeration Tank Widths").

The size of air bubbles is important. Mr. Lockett reached the conclusion, as a result of some experiments with Manchester sewage, that diffused air is approximately three times as effective as air applied in the form of large bubbles such as obtain with plain tube aeration.[17] It was thought formerly that the size of bubble from diffuser plates varied inversely as the permeability of the diffuser plate, permeability being defined as that quantity of dry free air, in cubic feet per minute, which will pass through 1 sq ft of dry plate at 70° F, and 30 in. of mercury absolute, under a pressure equivalent to 2 in. of water. For this reason plates of a relatively low permeability (10 to 25) and with relatively high frictional resistance (8 to 12 in. of water with 2 cu ft per min per sq ft of wet plate) were used. It has been found, however, that permeabilities of between 30 and 40 and even as much as 50 are satisfactory, and that there is little difference in the size of bubbles diffused from plates of 15 permeability and those diffused from plates of 60 permeability. Mr. Arnold J. Beck, of the Sanitary District of Chicago, has found that the average bubble diameter diffused from forty permeability plates under normal operating conditions is 0.0813 in., having a volume of 0.00028 cu in.[18] The plates of the higher permeability not only have lower frictional resistances (4± in. of water), but also do not clog as readily as the plates of lower permeability.

Plate dimensions have been standardized at 12 in. by 12 in., with thicknesses from 1 to 1.5 in.; however, it is understood that plates 18 in. square are being considered for large installations.

The diffuser plates may be set in concrete or in metal plate containers, or they may be set directly into concrete air channels in the bottom of the aeration tanks. Plate containers holding as many as eight to ten individual plates are in common use. Whether plate containers are used, or whether the plates are set directly into channels formed at the bottom of the tanks, the

[17] "Activated Sludge Process: Aeration and Circulation," by William T. Lockett, *Proceedings*, Assoc. of Managers of Sewage Disposal Works, 1925, p. 94.

[18] Paper presented before the Eighth Annual Convention of the Central States Sewage Works Assoc., Urbana, Ill., October 25, 1935. (Not published.)

bottoms of the aeration tanks should be poured flat and the containers or concrete channels built up in the bottoms of the tanks. Plate containers are favored on large installations; however, with careful workmanship in the forming and pouring of concrete plate channels, plates may be satisfactorily set into the bottoms of the tanks without the aid of plate containers.

Plate containers have usually been of pre-cast concrete sometimes with non-rusting metal edges next to the plates, or with metal supports between the plates, or both. A cast-aluminum container, recently developed, is now being used. Cement grout is the generally accepted jointing material; however, the plates cannot be readily removed where a cement grout is used. The use of rubber gaskets and wedges with clamps for holding the plates in place offers possibilities in the direction of a jointing material which will allow easy removal of the plates. Rubber gaskets should be corrugated so as to obtain a good air seal which is not obtainable with a plain rubber gasket. A special rubber has been developed for use in sewage.

Diffusion tubes have been used on several smaller installations. These tubes have the advantages of being easy to remove and clean without draining the aeration tank and of utilizing the economy of deep tank construction and, at the same time, of being placed at relatively shallow depths, thereby making a saving in the cost of air compression. The disadvantages of diffuser tubes are the higher cost than the cost of plates for a given air capacity and the unevenness of air dispersion due to the fact that the air tends to escape through the top of the tube.

The air used in diffuser plates or tubes should be filtered so as to remove any foreign matter that would tend to clog the diffusers; air compressors, air piping, and plate containers should be of materials and construction such as not to permit oil, rust, or flaking from metal to get into the air stream.

Air Piping.—The main object to be attained in the design of air piping is a uniformity of air pressure over the entire aeration tank with due regard to economic balance between installation and compression costs. In general, this will mean over-sized pipe rather than the economic size of pipe computed strictly in accordance with the air-flow formula.[19] Losses in heads in valves are conveniently computed on the basis of the additional length of straight pipe to give an equivalent loss (see tables in compressed air handbooks). Although air-valves are almost universally placed on the feeder pipes to a group of diffuser plates, these valves should not be depended upon for close regulation of air pressure. Standard angle-valves are usually all that are required for feeder pipes to each plate container, or group of containers, although needle-valves have been found convenient in small plants.

In some large plants air mains are run longitudinally along the aeration channels 4 or 5 ft above the bottom of the channels, each main serving two channels with laterals through the baffle-walls or the division walls; generally, however, the mains are carried on top of the V-shaped baffle or division mains with each main serving two channels. The two methods are illustrated in Fig. 5. The question as to which system to use is largely a matter of cost in

[19] For a comparison of various formulas see "Air-Pressure Losses in Piping of Activated-Sludge Plants," by Henry L. McMillan, M. Am. Soc. C. E., *Engineering News-Record*, Vol. 91, No. 5, August 2, 1923.

each particular case; however, the mains placed down into the channel have the disadvantage of creating eddies in the spiral flow of the liquor and of increasing the tank friction. In so far as possible the air piping should be designed and housed so as to conserve the heat in the air, inasmuch as the volume of the air delivered to the plates and the work done by it merely as a mixing agent is directly proportional to the absolute temperature of the air. Where the plate containers are poured in place on the bottom of the tanks and where the air channel beneath the plates is continuous and sufficiently large, the number of air-pipe laterals (down pipes) may be considerably reduced, or even eliminated, by making connections at each end of the air channel beneath the plates. The use of a continuous air channel also assists in the drainage of the air channels beneath the plates. Where a continuous channel is used the plates should be distributed uniformly along the channel with reference to permeability.

Piping may be of cast iron, galvanized iron, or steel, galvanized iron being preferred for small sizes (4 in., or less), cast iron for intermediate sizes (6 in. to 36 in.), and steel for larger sizes, although this is a matter of wide variation. Inasmuch as the air piping is subjected to differences in temperatures of as much as 200° F, ample expansion joints should be provided. In general, bell-and-spigot joints on cast-iron pipe and flexible couplings on steel pipe will provide for contraction and expansion.

Method of Returning Sludge.—The sludge should be returned uniformly from the clarifiers (that is, the same percentage should be returned) to each aeration tank battery. This may be obtained: (1) By mixing all the return sludge and raw sewage for the entire plant in a mixing channel before the mixed liquor goes into the aeration batteries; (2) by dividing the plant into a number of units which will accomplish the same result as under Method (1); or (3) by measuring or proportioning devices which will insure the correct ratios of sewage and sludge being drawn into each aeration battery from a common sewage-feed channel and a common return-sludge channel.

Many of the early activated sludge plants (Manchester, Worcester, Houston, and others) have air-lifts for returning sludge; however, most of the large plants built in the United States since the Houston plant have been equipped with centrifugal pumps for returning sludge. The personnel at the Chicago Sanitary District has obtained, by experimentation, a sufficiently efficient air-lift to meet its particular needs at the new Southwest Sewage Treatment Works and has incorporated sludge return air-lifts in the plans for the Southwest Treatment Plant. Again, this is a matter of cost, although the air-lifts no doubt accomplish some aeration. Although pumps have some advantage over air-lifts in the matter of efficiency, in a large plant, the cost of, and the head losses in, the piping or channels (or both), used in getting the sludge from the clarifiers to the pumps and back to the aeration tanks, may more than offset the efficiency advantage of the pumps. In a large plant the air-lifts may be placed between one or more aeration tank batteries and the clarifiers serving them so as to obtain a maximum of simplicity in sludge-return piping.

Measuring Devices.—However small or large the plant, the aeration tanks and appurtenances should be equipped with devices for measuring sewage flow, power, air, return and excess sludge, temperatures of air, sewage, and

atmosphere, and air pressures. If the plant consists of more than one aeration tank battery, either measuring or proportioning devices should be provided for assuring an equable flow of sewage, return sludge, and air to each aeration tank. Measuring devices of the Venturi tube type have proved satisfactory for sewage, return sludge, and air. In many plants the return sludge is mixed with the raw sewage before subdividing the flow of mixed liquor into the individual aeration tank batteries, adjustable weirs being used either at the inlet or at the outlet of the aeration tanks to secure an equal distribution to the different tanks. Air-valves alone are usually depended upon to secure an even distribution of air.

Conclusions

The curve shown as Fig. 2 is offered as an attempt to arrive at a practical guide to be used in approximating the detention period and percentage of return sludge to be provided for in the design of aeration tanks. It is questionable whether the curve is applicable to activated sludge plants treating certain trade wastes with unusually high ratios of bio-chemical oxygen demand to suspended solids. The plant-operating data from which the curve was plotted indicate that the detention period and percentage of return sludge and, consequently, the capacity of the aeration tanks, should be largely proportional to the strength of the sewage to be treated. In the writer's opinion, this is in accordance with the theoretical considerations involved.

Once the capacity of the aeration tanks is determined, details of the tank design are largely a matter of the economic balancing of costs. Improvements in the design of aeration-tank accessories are being made continuously.

Acknowledgments

Information to support the views offered in this paper was secured from various sources, for the most part, as cited by footnote references. However, special acknowledgment is due to: L. C. Whittemore, M. Am. Soc. C. E., for data pertaining to the Chicago North Side Plant; Darwin W. Townsend, M. Am. Soc. C. E., for data observed at the Milwaukee Plant; and C. K. Calvert, Affiliate, Am. Soc. C. E., for data relative to the Sewage Treatment Plant at Indianapolis.

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DISCUSSIONS

SELECTION OF MATERIALS FOR ROLLED-FILL EARTH DAMS

Discussion

By Charles H. Lee, M. Am. Soc. C. E.

Charles H. Lee,[37] M. Am. Soc. C. E. (by letter).[37a]—The generous response in discussion has fulfilled the writer's hope that the subject-matter of this paper might be expanded. All aspects of the subject have received attention, including a number of points not listed in the paper. It is regretted that the scope of the latter precluded consideration of such broader features of earth dam design and construction as were mentioned by Mr. Floris. This subject has been covered recently, however, in a very able manner by T. T. Knappen, M. Am. Soc. C. E.[16] With regard to the penetration needle mentioned in several discussions, the writer is familiar with it and considers it a very effective and useful tool.

The five principal requirements for suitability of impervious material, as listed by the writer, were accepted with more or less amplification by all who discussed the paper. Additional requirements pertaining to shape of particles, colloidal material, and cracking characteristics have been contributed. The discussions will be commented upon in the order that the subject-matter appears in the paper.

Cohesion.—It was pointed out by Mr. Baumann that cohesion due to the surface tension of water is absent under conditions of saturation where no particle surface is exposed to air. He recommends that all stability tests on impervious fill be conducted upon saturated material. The writer confirms this rule and would add that all samples that are small enough so that the density is affected by surface tension, should be submerged when

Note.—The paper by Charles H. Lee, M. Am. Soc. C. E., was published in September, 1936, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: December, 1936, by Messrs. T. T. Knappen, and Paul Bauman; January, 1937, by Messrs. William C. Hill, A. Floris, and Fred D. Pyle; February, 1937, by Messrs. Joel B. Cox, Stanley M. Dore, John E. Field, William P. Creager, and Joseph Jacobs; March, 1937, by Messrs. C. H. Kadie, Jr., and Ralph Bennett; April, 1937, by Ralph R. Proctor, M. Am. Soc. C. E.; and May, 1937, by L. F. Harza, M. Am. Soc. C. E.

[37] Cons. Hydr. Engr., San Francisco, Calif.

[37a] Received by the Secretary August 18, 1937.

[16] "Calculation of the Stability of Earth Dams," presented before the World Power Conference, Washington, D. C., September, 1936.

under test. This is the practice in making the Terzaghi consolidation test. It might also apply under certain conditions to shear tests.

Although open-minded with respect to the relation of thin films of water to cohesion, Mr. Dore agrees that the latter is always accompanied by the presence of "fines" and that its degree varies with the degree of fineness of particles and the percentage of "fines". In this connection, it may be stated that at all atmospheric temperatures water is present on the surface of "fines", in the form by hygroscopic moisture. Such moisture is in equilibrium with, and is derived from, water vapor in the air. Water, therefore, goes "hand in hand" with "fines", and there is the presumption that they both have some relation to cohesion.

Grading.—There is general agreement that well graded materials are ideal for the construction of the impervious sections of rolled-fill earth dams. The Talbot grading formula is also recognized as a satisfactory criterion for selection, provided n is confined to certain narrow limits. Mr. Jacobs submits statistical analyses from which he concludes that the value of n should be within the range of 0.25 to 0.40. This slightly widens the limit proposed by the writer, which was 0.25 to 0.33. As stated by Mr. Jacobs, the lesser values of n are more desirable on account of better workability of material. With the present tendency to increase the weight of rolling equipment and effectiveness of processing in compaction, however, increasing the value of the upper limit of n to 0.40 may not be inadvisable. Mr. Hill points out that the Talbot curve with $n = 0.33$ falls along the middle of the stabilization band defined by the Minnesota Highway Department for wearing surfaces for gravel roads.

Mr. Jacobs believes that the degree of conformity of a mechanical analysis curve with the Talbot grading curve within the specified limits of n is a dependable indication of potential density. Mr. Cox disagrees with this conclusion. He contributes interesting information regarding the abnormally high porosity of Hawaiian laterite soils which he attributes to the expanded character of the colloidal fraction. The condition described by Mr. Cox probably results from a high degree of flocculation in the soil induced by the chemical constituents of the local soil water. This condition is not confined to laterite soils or equatorial regions, but is encountered locally wherever soils of fine texture are highly flocculated. It may result from a variety of causes. One well known example is the soft mud deposited by fresh-water streams where discharging into salt water. Loess (wind-deposited) soils of fine texture may also have abnormally high porosity as deposited, although this may be readily reduced by heavy rolling. The latter is also true of highly porous cloudburst débris. Porous volcanic ash beds, on the other hand, may resist compaction. All such highly porous soils represent special conditions which must be taken into account, whether in dam, foundation, or other types of earth construction. The soils for which porosity and mechanical analyses were presented in Fig. 1, and also those with which Mr. Jacobs worked, are of the deflocculated type most common in earthwork in temperate zones.

Mr. Jacobs and Mr. Cox both question the value of the grading equation as an indicator of relative permeability. This subject will be discussed under the heading, "Permeability". Mr. Creager well describes the functions of the various fractions of a well graded soil mixture.

Compaction.—Mr. Jacobs compares the densities attained at present in compacted earth embankment and in mass concrete, and finds that as an average the latter attains a density 25% greater. He points out that they are composed of the same types of material and differ only in the degree of proportioning and mixing and in the method of compaction. His query as to whether maximum economic densities are being attained in earth mixtures, is a challenging one. The present tendency toward use of heavier rolling equipment, as illustrated by the work described by Mr. Baumann and referred to by Messrs. Kadie and Dore, will doubtless aid in reducing this difference. A more effective means will probably be found in the greater use of mechanical vibration equipment, as emphasized by Mr. Dore. It is also probable that, as pointed out by Mr. Pyle, there is a progressive compaction after completion of the dam resulting from the superimposed weight of material and expressed in settlement. No information is available as to the extent of such increase in density, but it is unquestionably appreciable especially in the lower courses. Mr. Pyle does well to suggest that compaction tests include loading commensurate with the weight of the dam. Mr. Hill's reference to the influence of higher temperature in producing greater density, as shown by Mr. Hogentogler's experiments, also has possibilities. This effect doubtless results from the lower viscosity of attached water resulting in increased lubrication.

Another field for investigation is the possibility of facilitating expulsion of air during the compacting process, thus reducing the air content at maximum compaction. The effectiveness of vibration in increasing density may be attributed in part to this.

Drainage.—Messrs. Knappen and Harza have contributed to this subject by emphasizing its importance in dams for flood control and other uses where impermeability is not essential. The permanence and stability of such dams depend largely upon adequate drainage. Natural drainage may be provided by use of materials that are progressively more permeable down stream. To be effective, drainage must prevent the building up of hydraulic pressure in any part of the dam to the point where the load is transferred from the solid material to water. It must also prevent the washing out of fine material. Mr. Harza does well to emphasize the importance of drainage for engineering structures.

Shape of Particles.—Messrs. Dore and Creager explain the function of individual particle shapes in providing resistance to movement in earth embankment. Sharp, angular particles develop greater internal friction than smooth, rounded ones and are distinctly advantageous. This is a characteristic to be taken advantage of, if possible.

Colloidal Fraction.—Messrs. Pyle and Cox emphasize the influence of appreciable quantities of colloidal material upon a soil in destroying the value of mechanical analysis as an index of character. Mr. Cox illus-

trates this phenomenon by reference to the Hawaiian laterite soils. This point is well taken and shows the value of extending mechanical analyses down at least to maximum clay size (0.005 mm) and further if possible.

Cracking Characteristics.—Messrs. Pyle and Field both refer to the undesirability of using material with an excess of fines, due to its tendency to form shrinkage cracks or planes of lessened resistance to water. Such material has a large capacity for water, and if the dam is unwatered for any length of time, cracks may form which will have disastrous effects upon refilling. This possibility is especially important in semi-arid and arid regions. The writer's minimum limiting curve is designed to exclude such material; but if a given material approaches this limit, has an appreciable colloidal fraction, or a high degree of flocculation, it is recommended that the standard Atterberg shrinkage tests be made before final selection.

Air Content at Maximum Compaction.—From his experience with Hawaiian soils, Mr. Cox considers the establishment of a permissible air-content at maximum compaction as a very important feature of selection of suitable materials for rolled-fill dams. His need for such a limit results from the prevalence in the Islands of highly flocculated soils with appreciable colloidal fraction. In Continental United States it is believed that, except in special cases, the choice of materials in conformity with

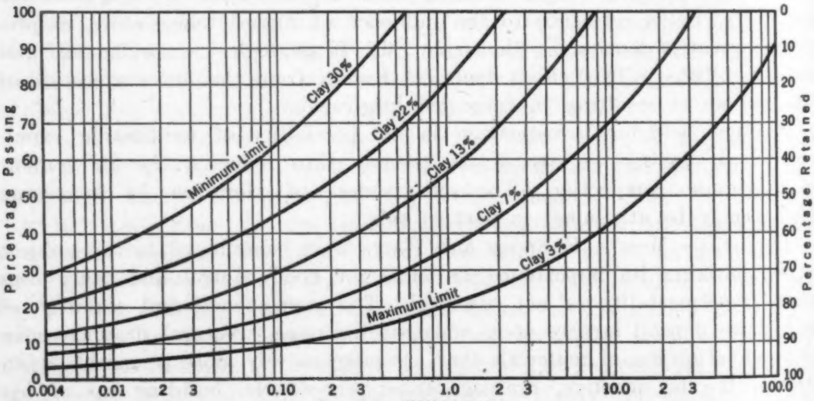

FIG. 23.—PROPOSED LIMITS IN MECHANICAL ANALYSIS FOR GRADED MATERIALS SUITABLE FOR IMPERVIOUS SECTIONS OF ROLLED-FILL DAMS (REVISED FORM OF FIG. 6).

the writer's limiting curves in Fig. 23 (which is a revision of Fig. 6) will result in securing a permissible air-content. As a tentative value, the writer uses 5% by volume, or 3.5% expressed as equivalent water in terms of dry weight. (See Fig. 4 and supporting tabulations.)

Impermeability.—This subject has received discussion from a variety of experiences and viewpoints. As pointed out by Mr. Harza, water conservation is of the greatest importance in the West and minimum leakage is there essential. In the more humid Eastern States, however, water has little value, and water-tightness is not essential. This is especially true of flood-control dams, of which many have been, or are being, built.

The following is a summary of stated quantitative limits for leakage through rolled-fill earth dams:

Writer: For water-tight dams, 0.10 gal per sq ft per day is stated as a limit; the maximum allowable leakage for any dam is 1.00 gal per sq ft per day.

Knappen: A satisfactory maximum allowable limit is 100 gal per sq ft per day, the value in any given case being largely a matter of economics.

Cox: The limiting value is a question of economic loss from leakage, and the safety of the structure.

Dore: For core material in water supply dams, 1 gal per sq ft per day is an acceptable limit; for hydro-electric power dams, 2 to 3 gal per sq ft per day is acceptable.

Jacobs: The limiting value is a question of the economic value of water loss and the possibility of piping. There are many entirely stable dams with seepage loss as great as 5 gal per sq ft per day. From economic value alone losses as great as 50 to 100 gal per sq ft per day, may be permissible.

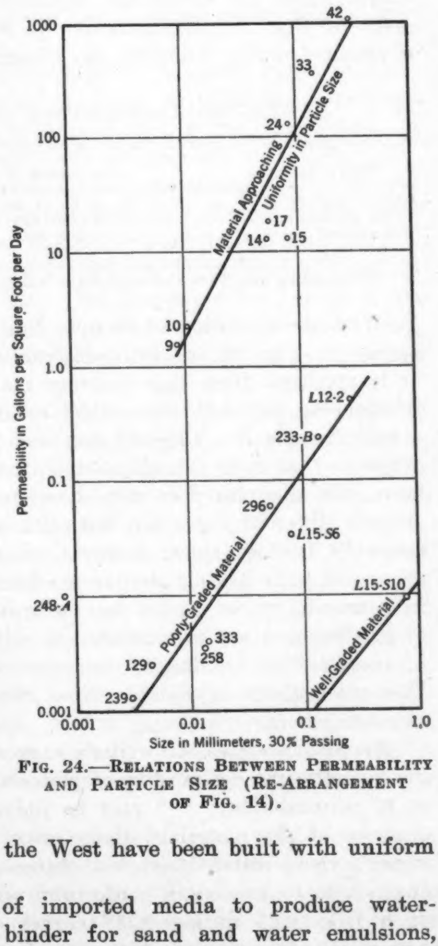

FIG. 24.—RELATIONS BETWEEN PERMEABILITY AND PARTICLE SIZE (RE-ARRANGEMENT OF FIG. 14).

Messrs. Baumann and Field describe ingenious uses of differing types of material to produce water-tightness with stability. These represent normal practices where limited material is available for impermeable construction. Where ample satisfactory material has been available, many impervious dams in the West have been built with uniform section.

Mr. Bennett refers to the use of imported media to produce water-tightness, such as chemicals, as a binder for sand and water emulsions, and oil admixtures for application to clay. The writer might add to this list the use of sodium carbonate or sodium chloride with loam soils as deflocculants to improve water-tightness of the back-fill in core trenches or core walls. Considerable experimental work has been done and actual use of such chemicals for this purpose has been made in several instances.

Messrs. Cox and Jacobs take issue with the writer's conclusion that there is a correlation between mechanical analysis and the permeability coefficient as determined from actual test. Mr. Cox submits data for seven samples of Hawaiian soils (Table 9) and in Fig. 14 compares diagrammatically the permeability and size, in millimeters (30% passing), including in the latter study both these samples and twelve samples for which data are given by the writer (see Fig. 5). To supplement the mechanical analysis data shown in Table 9, Mr. Cox has kindly furnished the complete curves which the writer has replatted in Fig. 23. Fig. 24 is a re-arrangement of Fig. 14. Examination of the mechanical analysis curves for the nineteen samples indicates the classification shown in Table 14.

TABLE 14.—CLASSIFICATION OF SAMPLE PLOTTED IN FIG. 23

Type of material	Lee Sample Nos.	Hawaiian Sample Nos.
Uniform texture........	9, 10, 14, 15, 17, 24, 13, 42	258
Poorly graded..........	*L*15–*S*6, *L*12–2	333, 296, 233*B*, 239, 248-*A*, 129
Well graded*...........	*L*15–*S*10	....

* Conforming with Talbot equation for $n = 0.25$ to $n = 0.33$.

With the exception of Sample Nos. 248–*A* and 258 these same groupings appear on Fig. 24 and lines designated correspondingly are shown thereon. It is apparent from this diagram that for each grading group there is a relationship between mechanical analysis and permeability and that the reason Sample No. *L*15–*S*10 has such low permeability in comparison with others is that it is the only sample having well graded material. Furthermore, the diagram does not show "enormous difference between soils of entirely different character, but with somewhat similar mechanical analysis curves." It does show, however, enormous differences between dissimilar groups of soils having similar mechanical analysis curves. The differences in permeability, as shown by the grouping, are apparently due to degree of grading and not to character of soil. Within any group, the differences in permeability are due to character of soil as controlled by particle size. The eccentricity of Sample Nos. 248–*A* and 258 may be due to observational errors.

Mr. Jacobs states the writer's suggestion (which he questions) to be that the "qualitative desirability of materials for earth dam construction, * * * as to permeability * * * may be judged by determining, from mechanical analyses of the materials, their value of n in the Talbot formula." The writer's view, restated, is that materials which are shown by mechanical analysis to be graded in conformity with the Talbot equation, with values of n from 0.25 to $n = 0.33$ (curves concave upward), have much lower permeability coefficients than poorly graded materials (sloping curves straight or irregular) and very much lower than uniform textured materials (curves convex upward or vertical). This statement, it is believed, is confirmed by Fig. 24. With regard to Mr. Jacob's proposed limits of $n = 0.25$ to $n = 0.40$ and "effective size" not to exceed 0.5 mm, this con-

dition would be fulfilled by Fig. 6 or Fig. 23 if the maximum limiting curve were drawn for $n = 0.40$ instead of $n = 0.33$. As previously stated, such a change may be desirable in the near future in view of the improvement in field compaction methods.

The writer agrees with Mr. Jacobs in questioning the accuracy of present empirical formulas for ground-water flow as applied to earth mixtures used in construction. As he well states, "permeability formulas have been devised for a more or less uniform and a relatively fine material and were not intended for such graded mixtures as are customary for earth dams." For many years the writer has depended upon actual permeability tests rather than the use of formulas.

Mechanical Analysis.—Almost every discussion included comment upon the writer's proposed limits for grading materials suitable for impervious sections of rolled-fill earth dams, as set forth in Fig. 6 and Fig. 7 and the supporting tabulations. These comments are summarized as follows:

Mr. Knappen prefaced his comments by the statement that "greatest economy generally results from the use of materials closest to the site." He considers that materials finer than the writer's minimum limit may be used for impervious core construction, dependence being placed upon the outer shell for structural strength; also that materials coarser than the maximum limit may be used where the water has little value, where large leakage is permissible, and where care is taken in design to pass the water without injury to the structure.

Mr. Baumann states that material which is not initially suitable because of gradation defects can be made so by proper compaction. The writer would point out, however, that this statement applies principally to material, the large pieces of which are readily crushed in rolling, such as that used in Zone 3 of San Gabriel Dam No. 1.

Mr. Dore considers the minimum limit satisfactory, but that curves of suitable materials especially of glacial origin, with little clay, may lie outside the maximum limit for short distances.

Mr. Creager thinks that the limits are helpful for preliminary studies provided their use is guided by experience and judgment. He believes that some materials outside the limits might be suitable and that other materials within the limits might be unsuitable. For a more accurate definition of requirements for graded materials, he suggests intermediate lower limits with varying clay content, and would eliminate entirely the diagram for ungraded materials (see Fig. 7).

Mr. Jacobs defines limiting maximum curves as those with n values in the Talbot equation between 0.25 and 0.40 and an effective size not to exceed 0.5 mm. The percentage of fines (0.05 mm) under these limits would generally not exceed 15 per cent. The maximum limit ($n = 0.40$) for the 5-in. maximum size of particle would slightly exceed the writer's maximum limit (see Fig. 23).

Mr. Kadie considers the limits as too narrow, believing that the design of a dam should be predicated upon the available material. Mr. Proctor states that he does not use limits, placing dependence upon laboratory

compaction and permeability tests, and plasticity needle penetration tests in the field. Mr. Harza thinks that the need for attaining the ideal grading depends upon a number of factors and that no fixed limits can be established.

The use of limiting curves as a guide in studying mechanical analysis data is thus little favored, being generally confined to those familiar with dam construction for water supply storage.

Mr. Proctor and others point out that selection on the basis of mechanical analysis is at best preliminary and that other considerations, including the results of compaction and permeability tests, must form the basis for final selection. This is in accord with the writer's views as expressed in the paper. The principal field for selection by mechanical analysis, however, is in reconnaissance, preliminary investigation, general studies, small projects, etc., or where limited funds are available for investigation. Mechanical analysis is a widely known standard test of broad usefulness which can be performed at moderate cost by a variety of established laboratories. For these reasons the writer believes that there is utility for a set of limiting curves, as shown in Fig. 23.

In preparing these revised curves, the suggestion of Mr. Creager has been followed. Three intermediate fine limiting curves have been drawn from the control data listed in Table 15. In using Fig. 23 it will be

TABLE 15.—CONTROL DATA FOR INTERMEDIATE FINE LIMITING CURVES (SEE FIG. 23)

Curve	Maximum size of particle, in millimeters	Value of *n* in the Talbot equation	Percentage of clay
Minimum limit: No. 28 sieve	0.6	0.25	30
Minimum limit: No. 8 sieve	2.36	0.25	22
Minimum limit: $\frac{5}{16}$ in.	8	0.28	13
Minimum limit: $1\frac{1}{4}$ in.	32	0.30	7
Maximum limit: 5 in.	127	0.33	3

found that the most suitable materials are those which have mechanical analyses curves that lie entirely between adjacent pairs of curves.

With the addition of the intermediate curves in Fig. 23, the writer agrees with Mr. Creager that there is no practical need for Fig. 7, giving limits for ungraded materials, such limits being provided by the spread between the adjacent intermediate curves in Fig. 23.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

STRESSES AROUND CIRCULAR HOLES IN DAMS AND BUTTRESSES

Discussion

BY I. K. SILVERMAN, JUN. AM. SOC. C. E.

I. K. SILVERMAN,[39] JUN. AM. SOC. C. E. (by letter).[39a]—In a very constructive discussion, Mr. Mindlin shows that the stresses due to gravity in a solid two-dimensional homogeneous body containing holes involve the elastic constants of the material. His analysis yields Equations (60) which are to be compared with the Equations (34) of the paper. The appearance of the elastic constants in Equations (60) is analogous to their appearance in problems dealing with statically indeterminate systems in the theory of structures. Thus, Equations (34) may be considered to be a solution of a statically indeterminate system by methods applicable only to a determinate system. The maximum difference involved in these formulas from the point of view of pounds per square inch of stress can be determined by using $\nu = 0.18$ for Poisson's ratio. The maximum absolute value of this difference is, $c\, r_h \left[1 - \frac{3 - \nu}{2} \right] = 1.71$ lb per sq in. Practically, System II can be neglected entirely, the effect of mass forces being given by Equations (31). This fact suggests that in making photo-elastic tests for mass forces boundary stresses to be applied at the surfaces of the model could be computed by means of Equations (23), (24), and (25), after the stresses defined by Equations (26) are subtracted. Equations (60) are checked both by Equations (70) in the discussion by Mr. Brahtz and by Equations (77) in Mr. Fedorov's contribution.

Mr. Vetter presents Equation (106*b*) to be compared with Equations (18) and (31) of the paper, and shows that the numerical results of Table 3 coincide very closely with those given in Table 1. His reference to the

NOTE.—The paper by I. K. Silverman, Jun. Am. Soc. C. E., was published in November 1936, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: February, 1937, by Messrs. R. D. Mindlin, and Chesley J. Posey; April, 1937, by Messrs. J. H. A. Brahtz, V. L. Fedorov, and F. W. Hanna; May, 1937, by C. P. Vetter, M. Am. Soc. C. E.; and June, 1937, by Fred L. Plummer, M. Am. Soc. C. E.

[39] With U. S. Bureau of Reclamation, Denver, Colo.

[39a] Received by the Secretary August 25, 1937.

"unwieldy" equations of the paper is not clear. Apparently, he is referring to Equation (18) and Equation (31) which give the hoop stresses at $\rho = r_h$ for water and mass, respectively. These equations are of the simplest type to apply. They contain trigonometric terms and terms which involve the constants of the structure; that is, the angle of batter, the location of the center of the hole, etc. They can be applied without any previous knowledge of the theory of elasticity or of the steps involved in their derivation.

Equation (106*b*), which gives the corresponding hoop stresses, involves the following steps: (1) Determination of the state of stress at the center of the hole; (2) determination of the direction of the principal stresses at the same point; (3) determination of the principal stresses, s_1 and s_2; and, (4) substitution of the various values of the argument, θ_1, in Equation (106*b*). Step (4) involves the use of a new set of co-ordinate axes from which the values of θ_1 are to be measured. In the application illustrated in the paper this step involved a rotation of $-7° 38'$ with respect to the natural set of axes taken along the sloping face.

For the sake of completeness the equations required for these operations are given for the case of water loading:

$$s_x = \frac{p\,x}{K^2} - \frac{2\,p\,y}{K^3} \qquad (109a)$$

$$s_y = -\,p\,x \qquad (109b)$$

and,

$$s_s = -\,\frac{p\,y}{K^2} \qquad (109c)$$

$$\tan 2\,\epsilon = \frac{2 s_s}{s_x - s_y} \qquad (110)$$

$$s_1 = \frac{s_x + s_y}{2} + \sqrt{\left(\frac{s_x - s_y}{2}\right)^2 + s_s^2} \qquad (111a)$$

and,

$$s_2 = \frac{s_x + s_y}{2} - \sqrt{\left(\frac{s_x - s_y}{2}\right)^2 + s_s^2} \qquad (111b)$$

It is seen that an intimate knowledge of the theory of elasticity is required for the correct application of Equation (106*b*) which is simple in form but complex in use.

Mr. Vetter states that the writer has avoided the necessity for the assumption that the hole is very small as compared with the other dimensions of the body in using Equations (42) to (46). The point of the solution, however, is such that this assumption is not needed. The following fact is responsible for the mathematical solution of the problem; Equation (42), the general solution for plane stress problems in polar co-ordinates, yields stresses according to definitions, Equation (41), which are of the identical form of Equations (10) to (12), inclusive. In applying the method of Filon to an annular disk, the ratio of the outer diameter to the inner

diameter is involved. For practical purposes, and in agreement with St. Venant's principle, this ratio is taken as zero, and the assumption is made at the proper point. (See text following Table 2.) That this assumption did not appear in the main body of the paper, as Mr. Vetter points out, is due to editorial exigencies, which required that the greater portion of the mathematics involved be placed in an Appendix. In a general way this assumption is implied in non-mathematical language in the first sentence of the opening paragraph of the section entitled "General Formulas."

Mr. Vetter refers to the paucity of references. Footnote references (3) and (4)[39b] were deemed to be sufficient cross-references to all work involving stresses around discontinuities.

Professor Plummer questions the assumption of linear stress distribution as given by the Lévy solution, Equations (8) and (22). Both[40] experiment and theory have shown that this departure from linear behavior is confined to the lower third portion of a triangular dam. The maximum possible discrepancy occurs at the boundaries and has very little effect on the principal stresses involved in the region of the hole. The writer believes that the ordinary engineering theory of straight-line variation is sufficiently accurate for all practical purposes, especially when such complex questions of temperature, shrinkage, etc., are equally involved.

Professor Plummer states that his experimental results are quite different from those obtained by using the writer's theoretical equations. Mr. Vetter has shown that comparable results can be obtained by determining the principal stresses at the center of the hole and applying Equation (106b). Experiments[41] made at the U. S. Bureau of Reclamation show a very close agreement between experimental and theoretical results. It is not to be expected that the models used by Professor Plummer will give any conclusive experimental results. Probably the maximum size of model possible in a photoelastic testing apparatus is of the magnitude of 6 in. The diameter of the hole representing a gallery of two-one-hundredths of the height would be approximately 0.12 in. The difficulties in measuring stresses around holes of this size are apparent.

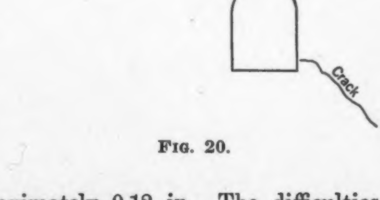

FIG. 20.

[39b] *Proceedings*, Am. Soc. C. E., November 1936, pp. 1361–1362.

[40] *Transactions*, Am. Soc. C. E., Vol. 96 (1932), pp. 489–591; and Vol. 101 (1936), pp. 1258–1259.

[41] Memorandum to Chf. Designing Engr.; "Design Data for Gallery Stresses as Determined by Plaster-Celite Models," by F. M. Russell, Jun. Am. Soc. C. E.; and "Gallery Stresses and Steel Reinforcement," by C. N. Zanger and J. H. A. Brahtz, *Technical Memorandum No. 451.*

The type of model used at the U. S. Bureau of Reclamation consists[42] "of a square slab of plaster celite about seven times the width of the gallery opening. * * * The sides of the square are parallel to the direction of the principal stresses occurring at the center of the area." The slab was roughly 3 ft square.

Mr. Hanna summarizes the formulas in use by designing engineers and describes cracks in dams which he has observed. In a memorandum to J. L. Savage, M. Am. Soc. C. E., Chief Designing Engineer, U. S. Bureau of Reclamation, H. M. Westergaard, M. Am. Soc. C. E., describes cracks observed in dams and mentions a type of crack in buttresses of hollow dams (Fig. 20) which can be explained by the theory as given in this paper. The cracks occur in regions of tensile stress as predicted by Equation (18). In these regions the reinforcement has pulled out, which supports Professor Posey's contention that "the proper placing of steel bars around holes is of great importance."

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[42] "Gallery Stresses as Determined by Means of Plaster Celite Models," by F. M. Russell, Jun. Am. Soc. C. E.

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DISCUSSIONS

RAINFALL INTENSITIES AND FREQUENCIES

Discussion

BY MESSRS. C. S. JARVIS, AND A. J. SCHAFMAYER

C. S. JARVIS,[39] M. AM. SOC. C. E. (by letter).[39a]—The persistency of the straight-line relations as shown in Fig. 2 is so striking as to invite further consideration. It is found that the data included in Meyer's Table 15, Groups 2, 3, and 4,[40] conform moderately well to straight lines for the lower rainfall intensities (Fig. 15). However, for Group 1 the divergence of the data from straight-line trends becomes quite marked, thus requiring the use of irregular curves. Similar divergence from the straight-line relation is apparent likewise in the data furnished for Boston, Mass., by Charles W. Sherman,[41] M. Am. Soc. C. E., and re-plotted on semi-logarithmic paper in Fig. 16(*a*).

When the data from Group 3 of Meyer's Table 15 were plotted similarly on the same scale as that used for Fig. 2 of the paper, their inclination to fall on straight lines was quite pronounced; but when plotted on a much larger scale, the divergences became proportionately more apparent.

Approaching the problem independently, the writer undertook to analyze the excessive precipitation data[42] for Indianapolis, Terre Haute, Evansville, Fort Wayne, and Royal Center, in Indiana; Cleveland, Columbus, Dayton, and Cincinnati, in Ohio; Pittsburgh, Pa.; St. Louis, Mo.; and Chicago, Ill., representing a total of 301 yr of record, or an average of 25 yr per station, the maximum record period being about 34 yr for Indianapolis, with 33 yr for Pittsburgh, Columbus, and Cincinnati. The resulting intensity-frequency graphs are shown in Fig. 16(*b*), on which the data from 301 station-yr of composite record are summarized graphically, all conforming more or less closely to straight-line trends.

NOTE.—The paper by A. J. Schafmayer, M. Am. Soc. C. E., and the late B. E. Grant, Esq., was published in February, 1937, *Proceedings*. Discussion on this paper has been published in *Proceedings*, as follows: April, 1937, by Messrs. Victor L. Cochrane, and L. K. Sherman; June, 1937, by Messrs. J. O. Jones, Charles W. Sherman, Glen N. Cox, Garrett B. Drummond, Eugene L. Grant, Adolph F. Meyer, and Clinton L. Bogart; and September, 1937, by C. H. Eiffert, M. Am. Soc. C. E.

[39] Hydr. Engr., Soil Conservation Service, Washington, D. C.

[39a] Received by the Secretary August 25, 1937.

[40] "Elements of Hydrology," by Adolph F. Meyer, 1917, pp. 181–184.

[41] *Transactions*, Am. Soc. C. E., Vol. 95 (1931), p. 956.

[42] *Bulletin W*, U. S. Weather Bureau, 1930.

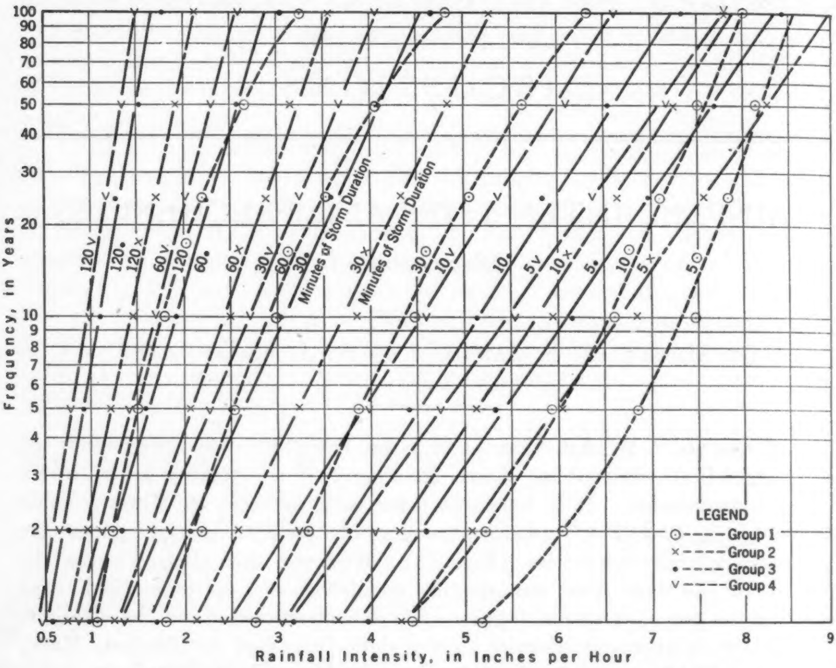

FIG. 15.—COMPARISON OF RAINFALL FREQUENCY-INTENSITY RELATIONS.

In order to afford a comparison of data for Cincinnati and Chicago from record periods terminating in 1914 and 1930, respectively, Tables 19 and 20 were compiled. These tables show generally increased intensities for given frequencies, due to the inclusion of the more recent data. The

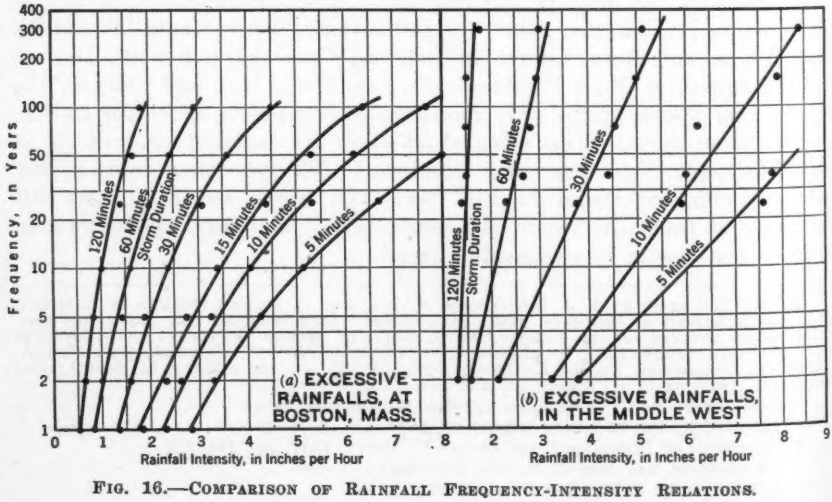

FIG. 16.—COMPARISON OF RAINFALL FREQUENCY-INTENSITY RELATIONS.

TABLE 19.—COMPARISON OF INTENSITY-FREQUENCY RELATIONS

Frequency years	5 min: Meyer*	5 min: U. S. Weather Bureau†	5 min: Authors‡	10 min: Meyer*	10 min: U. S. Weather Bureau†	10 min: Authors‡	30 min: Meyer*	30 min: U. S. Weather Bureau†	30 min: Authors‡	60 min: Meyer*	60 min: U. S. Weather Bureau†	60 min: Authors‡
				(a) Chicago, Illinois								
	....	0.37§	....	....	0.61§	....	....	1.18§	....	....	1.49§	....
2	0.36	0.44	0.40	0.64	0.68	0.63	1.05	1.29	1.10	1.28	1.63	1.34
5	0.45	0.51	0.50	0.75	0.77	0.78	1.25	1.45	1.40	1.45	1.87	1.74
10	0.55	0.55	0.55	0.85	0.82	0.89	1.35	1.55	1.62	1.60	2.05	2.05
				(b) Cincinnati, Ohio								
	....	0.35§	....	....	0.59§	....	....	1.33§	....	....	....	....
2	0.34	0.44	....	0.56	0.69	....	0.98	1.43	....	1.15	1.85	....
5	0.40	0.51	....	0.65	0.84	....	1.20	1.84	....	1.50	2.26	....
10	0.48	0.55	....	0.78	0.96	....	1.60	2.14	....	2.00	2.46	....

(Column heading: Precipitation Depths, in Inches, Exceeded Once in Given Number of Years During Designated Time; Time, in Minutes)

* Data from Table 14, pp. 178–179, Meyer's "Hydrology," 1917.
† Analysis of data from U. S. Weather Bureau *Bulletin W*, 1930, by C. S. Jarvis, as shown in Fig. 17.
‡ Data from Fig. 5, Schafmayer and Grant.
§ Derived from all pertinent data in U. S. Weather Bureau *Bulletin W*, instead of being limited to those higher values defined by the authors as "excessive rainfalls."

quantities in the columns headed, "Authors," were derived from Fig. 5, and show fairly close agreement between results of independent approaches by the authors and the writer toward rainfall frequency-intensity relationships for Chicago, even if the station-years of record considered were unequal, being 107 and 25, respectively.

TABLE 20.—DEPTHS, IN INCHES, REQUIRED TO QUALIFY AS EXCESSIVE PRECIPITATION FOR VARIOUS PERIODS

Authority	Duration of Storm, in Minutes: 5	10	15	30	60	100	120
U. S. Weather Bureau:							
Northern States	0.35	0.50	0.65	0.90	1.20	1.30	1.40
Southern States	0.40	0.50	0.60	0.90	1.50	2.30	2.70
The Authors, under heading, "Definitions of Terms"	0.35	0.50	0.65	0.90	1.20	1.30	1.40

Fig. 17 provides a graphic comparison of rainfall frequency-intensity trends for Cincinnati and Chicago, and indicates somewhat higher intensities for Cincinnati, except for 5-min periods, when the graph follows an irregular curve, crossing the straight-line trend for Chicago.

Whether the data from 12 stations, representing an aggregate record period of 301 yr, may be regarded and analyzed as a continuous record at one station for three centuries has never been settled authoritatively, although the assumption has been widely accepted. Valuable generalizations, trends, and conclusions have been derived from such composite records; but, thus far, no one has claimed and proved their equivalence to continuous records of equal lengths from single stations. Neither has it been established satisfactorily that straight-line relations inherently exist among observed hydrologic or other physical data, no matter what scale or form of chart is used. The natural inequalities and irregularities ap-

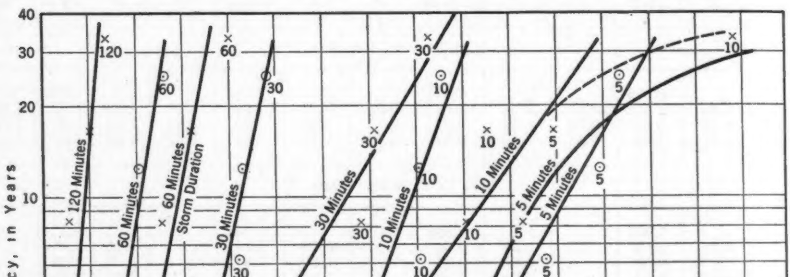

FIG. 17.—COMPARISON OF RAINFALL FREQUENCY-INTENSITY RELATIONS FOR CINCINNATI, OHIO, AND CHICAGO, ILLINOIS.

pearing in practically all basic data must be "ironed out" by some selective grouping, averaging, or other leveling process, or by plotting on small-scale charts, before the inherent deviations from the trend lines or curves are practically eliminated, or reduced to significant, readily understandable quantities, susceptible of logical interpretation.

The writer has been unable to reconcile the various trends of Fig. 2, whereby the rainfall intensities for given frequencies become slightly less severe as the station-years increase, although the added stations are supposed to be fairly comparable. One would naturally expect higher records to be developed as stations are increased in number and as the record periods are extended.

The authors have made important contributions to the subject of their paper, and have apparently established a safe basis for the design of storm sewers in the Chicago District.

A. J. SCHAFMAYER,[43] M. AM. SOC. C. E. (by letter).[43a]—The constructive type of discussion submitted brings out some interesting points. For example, Mr. Cochrane's comparison of Equations (18) and (27) in Table 8 shows a very remarkable agreement in intensities.

Mr. LeRoy K. Sherman questions the grouping of the Tennessee stations with the others included in the Chicago group. As pointed out by Professor Meyer, the writers are in agreement with his deduction that there is no material relationship between the intense precipitation for short periods of time and the quantity of annual rainfall.

As to Mr. Sherman's question: Are irregularities "ironed out" for small urban areas? the writers' study of the data led them to believe that irregularities are "ironed out" for areas requiring a time element of 15 min, or more, for excessive intensities. However, this may not be true for annual quantities, as shown by George W. Pickels,[44] M. Am. Soc. C. E. The writers made no attempt in this study to relate intensity and run-off or to secure simultaneous intensity and run-off data, as was done in the study by Messrs. Horner and Flynt.[13] It is evident, however, as stated by Mr. Sherman, that the intensity rates may be applied to run-off computation by the so-called rational method, or by the Sherman unit hydrograph[11, 12] method, or by empirical graphs, or tables of other types. Table 9, submitted by Mr. Sherman, shows the value of careful study in this part of the design.

In his studies of rainfall for Kansas, Professor Jones presented some interesting data. As he states, however, comparisons, especially with his ogive curves, are difficult. Although their studies led the writers to believe that the quantity of annual rainfall has no relation to the intensity for short durations, it appears that this is true only for the excessive rates with a high minimum intensity used by them, as shown under the heading, "Definitions of Terms." If lower intensities are included and a large number of storms of minor intensity are added to the tabulations, the principle of independence between intensity and annual precipitation is modified, and this independence tends to decrease as the minimum rate is reduced. For example, in Section (*a*) of Table 12, the occurrences listed by Professor Jones with intensities less than that used by the writers, number 1 077 in a total of 1 243 items, or more than 86%; but even with this increase in number of occurrences, the data as shown by the analysis of Professor Jones may be statistically comparable. Although such storms of minor intensity are of great importance from the standpoint of storage for agricultural or hydro-electric purposes, etc., they are of slight importance in the design of storm sewers and were not con-

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[43] Engr., Board of Local Impvts., City of Chicago, Chicago, Ill. Mr. Grant, co-author of this paper, died on October 30, 1936.

[43a] Received by the Secretary August 13, 1937.

[44] "Run-Off Investigation in Central Illinois," *Bulletin No. 232*, Eng. Experiment Station, Univ. of Illinois, pp. 91–93.

[13] "Relation Between Rainfall and Run-Off from Small Urban Areas," *Transactions*, Am. Soc. C. E., Vol. 101 (1936), p. 140.

[11] "Runoff from Rainfall by the Unit Graph," *Engineering News-Record*, April 7, 1932.

[12] "An Approach to Determinate Stream Flow," *Transactions*, Am. Soc. C. E., Vol. 100 (1935), p. 347.

sidered by the writers. The stations used were selected on the basis of the uniformity in the recorded data as shown in Table 4.

In Fig. 12 Professor Jones presents a chart showing the quantity-intensity relationship, since multiplying intensity by time gives the quantity or accumulated depth. The use of minutes in multiplying in this particular case gives a function of the quantity instead of the actual depth, but the ratio is constant. Professor Jones shows clearly the variations of the records of single storms from the Talbot type of formula. The writers also found that records of single storms showed considerable variation from the formula. However, the averages of a great many records of high intensity appear to fit very closely.

Mr. Charles W. Sherman brings up an important point. Geographical and topographical influences undoubtedly affect rainfall. The stations used, however, showed, by recorded data, a similarity of occurrences of high intensity under the existing geographical and topographical variations. It is as conceivable that several places more or less remote from each other should have similar rainfall intensity and frequency characteristics in the excessive range, while differing greatly in total annual rainfall, as that several such places might have the same altitude, even if hundreds of miles of mountains and valleys separate them geographically. The writers realized that the stations used are not "located within an area possessing uniform rainfall characteristics," as far as total annual rainfall is concerned. They believed, however, that these stations have very similar characteristics in the range of excessive intensities used in their investigations.

Meteorological cycles were considered in this study, but no attempt was made to determine the effects of such cycles except to assume that the data for the 659 station-yr and the 330 station-yr studies include approximately three complete registrations of the common or sun-spot cycle (in so far as it affects the high intensities considered). As to cycles of the order of hundreds of years, the writers agree with Mr. Sherman that the station-year method gives no indication of the effect of such cycles. More years of scientific recording than are now available will be necessary to determine their effect on rates of rainfall.

Conclusions (6) and (7) were based on the studies for the published paper and on unpublished studies of storm patterns which Mr. Grant had been making for some years prior to his death and which were somewhat similar to the studies by Frank A. Marston,[20] M. Am. Soc. C. E., referred to by Mr. Sherman. Because the Chicago gages are spaced irregularly and some are considerable distances apart, only fragmentary storm maps were secured in many cases. Only a comparatively few closed isohyetals near the storm centers could be developed, and most of the storms appeared to be much more irregular in shape and movement than those reported by Mr. Marston. Because these studies were incomplete, it was decided not to include them in the paper as presented. Therefore, no reference was

[20] "The Distribution of Intense Rainfall and Some Other Factors in the Design of Storm-Water Drains," *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 535.

made to the paper[20] by Mr. Marston. However, since this aspect of the general rainfall problem was noted in Conclusions (6) and (7), perhaps such a reference should have been made.

The inaccuracy of the tilting bucket type of record for very high intensities is pertinently noted by Professor Cox. Undoubtedly, the under-recording occurs principally during the periods of very high intensities. The statement quoted that "no correction factor was used for the precipitation corresponding to any duration", means that the writers applied no correction factor in using the published records. Wherever local observers, in preparing rainfall data for publication, may have distributed an excess, as shown by stick measurement over the automatic record, such distribution or correction, of course, was included in the published records, and was used by the writers. No study was made to determine the extent of such correction or distribution and no special consideration was given to it except to mention it in Item (*d*) under the heading, "Deficiencies in Data." A study was made for the Chicago District to determine the time of year when excessive rates occur. A chart showing these occurrences and their dates is included among the unpublished papers, and it can be summarized by stating that all the excessive rates for this District occurred between May 1 and October 21. The comments of Professor Cox as to the conditions affecting run-off, other than the rainfall rate, apply more directly to suburban or rural areas. In metropolitan sewer districts with large proportions of impervious areas those other factors have diminishing influences on the rate of run-off.

Mr. Drummond's charts, shown in Figs. 13 and 14, supply interesting information and should aid an engineer in determining whether the results of these studies can be applied to his local problem.

The statistical analyses of the results of these studies presented by Professor Eugene L. Grant, provide tests of validity of a different type than any considered by the writers. It is gratifying to note that they qualify under such tests. The important implication, as emphasized by Professor Grant, and again by Professor Meyer, is that, within the high intensity brackets used in these studies, stations many hundred miles apart and with dissimilar annual rainfall characteristics have similar frequency-intensity characteristics.

Mr. Bogert emphasizes the run-off factor in the final design of sizes and capacity of drains. The writer agrees with Mr. Bogert that these studies dealt with only one of the elements concerned. However, the determination of run-off is another step that requires a specific consideration of the items enumerated by Mr. Bogert for the particular locality as well as a considerable element of judgment.

The statement by Mr. Eiffert that judgment and caution should be used in applying the station-year method is undoubtedly true. It was the opinion of the writers, however, that the similar characteristics for the high intensities shown in the data assembled in their studies as well as in the studies by Meyer[4] justified using the stations selected. Mr. Eiffert's

[4] "Hydrology," by A. F. Meyer, John Wiley & Sons, 1928.

comments as to cycles as well as those of Mr. Charles W. Sherman are quite correct. Although the 19-station and the 10-station series in the paper include approximately three of the sun-spot cycles, they cannot show the effect of long-term cycles. Some of the older local stations and the two Chicago stations of the U. S. Weather Bureau, in the Chicago District series show more than one complete sun-spot cycle. The newer stations in Chicago are of such recent date that they do not cover even one of these cycles, as noted in the paper. It is very probable that further accumulation of local records will alter the local data somewhat. The writers were in complete agreement with Mr. Eiffert as to the need for more complete rainfall records.

As stated by Mr. Jarvis, the straight-line relationship is more apparent in some cases than in others. The writers noted, however, that the divergence appeared to decrease as the quantity of data increased. The variations from a straight line for the Group 1 and Boston data in Fig. 15 and Fig. 16 (*a*) are interesting and worthy of further study. The writers also found the greatest deviations from a straight line in the records of the 5-min and 10-min storms. Although the writers had observed the apparent arithmetical-geometrical progression relationships between the intensity and the corresponding time intervals, their studies were stimulated when the paper by Mr. Jarvis[6] was published in 1930.

Referring to the last part of Table 20, the Weather Bureau standard for excessive rates shown on the top line agrees with the writers' definition on the bottom line. Unless this is a typographical error, there must have been a new ruling since that of March, 1934, shown by the writers in detail, under the heading "New Rule of the United States Weather Bureau for Tabulation of Excessive Rainfall." The trend mentioned by Mr. Jarvis indicated in Fig. 2 may be partly due to the method followed by the writers. The 19-station series was developed first. Then the Chicago District was studied and tabulated, using the Weather Bureau stations, together with the City and Sanitary District stations. These latter stations included data from some storms of much higher intensities than were recorded on any of the local Weather Bureau records; for instance, the Mayfair gage of the City of Chicago in the storm of July 26, 1932, recorded 3.87 in. in a 2-hr storm, with diminishing intensities on adjacent gages and with only 1.06 in. recorded for a storm of 1.5 hours' duration at the official University Station of the Weather Bureau about 15 miles away. Hence, the averages for the Chicago District were higher than those for the nineteen original stations. Then, the ten stations whose average data were in closest correspondence with the Chicago data, were selected from the nineteen stations for the 330 station-yr series. This resulted in the averages of the ten stations being higher than those for the entire nineteen stations, but not as high as the averages for the Chicago District.

The study was prompted by the desire to develop a method for using the cumulative data from an increasing number of rain-gages and to in-

[6] *Transactions*, Am. Soc. C. E., Vol. 95 (1931), Fig. 3, p. 400.

vestigate the validity of the station-year method of using such data. The recent increase in the number of such gages in the Oklahoma[45] and Ohio[46] studies by the Federal Government may develop still further information on these and other points.

In conclusion, the writer wishes to express his appreciation to all who have contributed to the discussions and who have revealed important aspects and implications through them.

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[45] "The Life History of Rainstorms," Progress Report from the Oklahoma Climatic Research Center, by C. Warren Thornthwaite, *The Geographical Review*, Vol. XXVII, No. 1, January, 1937, p. 92.

[46] The Section of Climatology, Branch of Research, Soil Conservation Service, U. S. Dept. of Agriculture, is installing in the Muskingum Conservancy District in Southeastern Ohio 500 recording stations on 4½-mile quadrates over 8 000 sq miles. One district at Coshocton, Ohio, has gages on quadrates of ¼ mile over an area of approximately 6 000 acres.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

FLOW CHARACTERISTICS IN ELBOW DRAFT-TUBES

Discussion

BY HOWARD L. COOPER, ESQ.

HOWARD L. COOPER,[29] ESQ. (by letter).[29a]—The type of experimentation developed by Professor Mockmore will have a marked impression on the future of hydraulic machines and structures. This fact has certainly been demonstrated in the turbine and draft-tube laboratory of the U. S. Army Engineers, at Portland, Ore., in connection with the Bonneville Project. The writer wishes to discuss further some of the statements made by Professor Mockmore and to give a little more information on the draft-tube report, especially pertaining to the tubes that were tested after the work was finished at the Oregon State Laboratory and moved to the U. S. Engineer Laboratory, at Linnton, Ore.

Figs. 28 and 29, are submitted as evidence of the study that was started in an effort to find a relation between diameter of jet, radius of curvature, and angle of flare of a jet of water passing around a curve on a flat plate. Such a jet represents the natural flow as it probably offers less resistance than any other method of changing the direction of flow through 90 degrees. It is very easy to visualize the double spiral if one pictures this fan-shaped jet folded up from each side upon itself until it is confined in a 90° elbow. The major losses are then the energy required to form the double spiral and also the internal friction caused by the different strata of the whirl meeting with different relative velocities with respect to the initial flow.

The writer agrees with Professor Mockmore that Elbow No. 4 should be the most efficient, as the flattened shape conforms more nearly to the fan-shaped jet shown in Fig. 28 and Fig. 29. The double spiral is dissipated to a certain degree while making the turn. Fig. 30 shows a jet of water

NOTE.—The paper by C. A. Mockmore, M. Am. Soc. C. E., was published in February, 1937, *Proceedings*. Discussion on the paper has appeared in *Proceedings*, as follows: April, 1937, by F. T. Mavis, M. Am. Soc. C. E.; May, 1937, by Jerome Fee, Assoc. M. Am. Soc. C. E.; June, 1937, by Messrs. R. E. B. Sharp, and L. F. Harza; and September, 1937, by Ellery D. Fosdick, Esq.

[29] Office of Div. Engr., North Pacific Div., U. S. War Dept., Portland, Ore.

[29a] Received by the Secretary July 26, 1937.

passing around a standard elbow with the jet smaller than the elbow inlet; it can be clearly seen how one-half the double spiral begins to develop. Referring to the elbow studies, the writer is convinced that a slow-motion study of elbows, possibly using additional shapes, would show many inter-

FIG. 28.

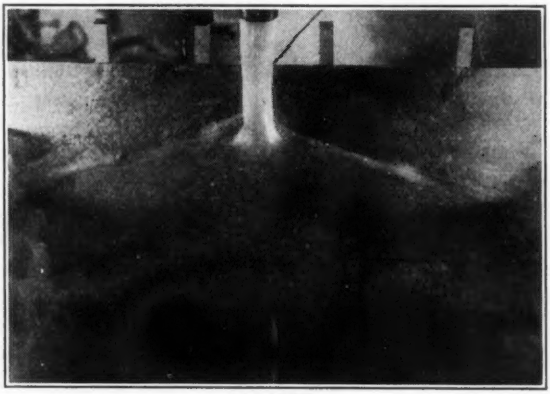

FIG. 29.

esting phenomena and possibly develop some ideas that would be a real contribution to the study of hydraulics.

Referring to the functions of a draft-tube, the writer wishes to take the liberty of discussing the advantages in a little more detail. The velocity of the water is reduced after leaving the turbine runner, but in doing so it

FIG. 30.

maintains a seal at all times and lowers the absolute pressure directly under the runner and, therefore, increases the total effective head on the unit. A poor draft-tube that does not maintain this seal loses this gain in velocity head and, therefore, reduces the output of the turbine, and also frequently causes serious vibration, which naturally reduces the life and mechanical efficiency of the unit.

The location of the center line of the runner above the tail-race is an important feature. Francis wheels can be placed much higher than those of the propeller type. A very careful study must be made of propeller type settings with respect to their elevation above tail-water, as the location depends on the proper combination of the plant σ, specific speed, ϕ, and the general design of scroll and draft-tube, in order to prevent cavitation at the higher loads and low tail-water conditions. ($\sigma =$ the cavitation coefficient = the ratio of the maximum possible pressure reduction within the runner without vaporization of the water, to the total effective head on the turbine. Plant σ is distinguished from Field σ by a suitable factor of safety. The ratio of the peripheral speed of the runner to $\sqrt{2\,g\,H} = \phi$.) Cavitation not only affects parts of the runner blades and throat liners, but sets up detrimental vibrations in the unit and the structure.

A draft-tube also should have enough velocity at the exit, together with the hydraulics of the tail-race, to keep the velocity high enough to clear the tail-race without the building up of undue head.

In large draft-tubes the writer makes the following recommendations based upon experimental observation. In addition to the horizontal splitter, at least two vertical splitters are suggested; also, the placing of six or more vertical, stream-lined struts in the exit half of the horizontal leg, the spacing to be arranged by experiment. These struts will cause the water to seal the exit with a uniform discharge and, therefore, increase the re-gain within the draft-tube.

TABLE 5.—HEAD, IN FEET, REQUIRED TO FORCE A FLOW OF 0.997 SECOND-FOOT THROUGH DRAFT-TUBE No. 2

Description	Angle of Whirling Component at Entrance	
	30°	40°
Without struts	0.4062	1.709
With struts, Arrangement No. 1	0.3971	1.695
With struts, Arrangement No. 2	0.3821	1.683

The information contained in Table 5 is taken from experimental data, Draft-Tube No. 2 being operated with, and without, the vertical struts in the horizontal leg near the exit (see Fig. 31).

Draft-tube passages that do not allow the water to flow smoothly reduce the re-gain in velocity head, and this method of experimentation, together with the use of slow motion pictures, possibly will enable most of the internal cross-currents to be checked and corrected, thereby increasing the re-gain in velocity head. The writer suggests that a material of practically neutral buoyancy be used in the water instead of the injection of air, because the air bubbles will flow toward the upper surfaces and will not give a true pattern of the flow. Besides, the air bubbles change the specific gravity of the liquid used, which change can not be conveniently

compensated, and no doubt will cause Pitot tube velocity readings to be incorrect.

After weeks of searching, the writer found a hardwood granular sawdust that would give a true pattern after being soaked in water for about 24 hr; it would neither float nor sink, and being practically the same weight as

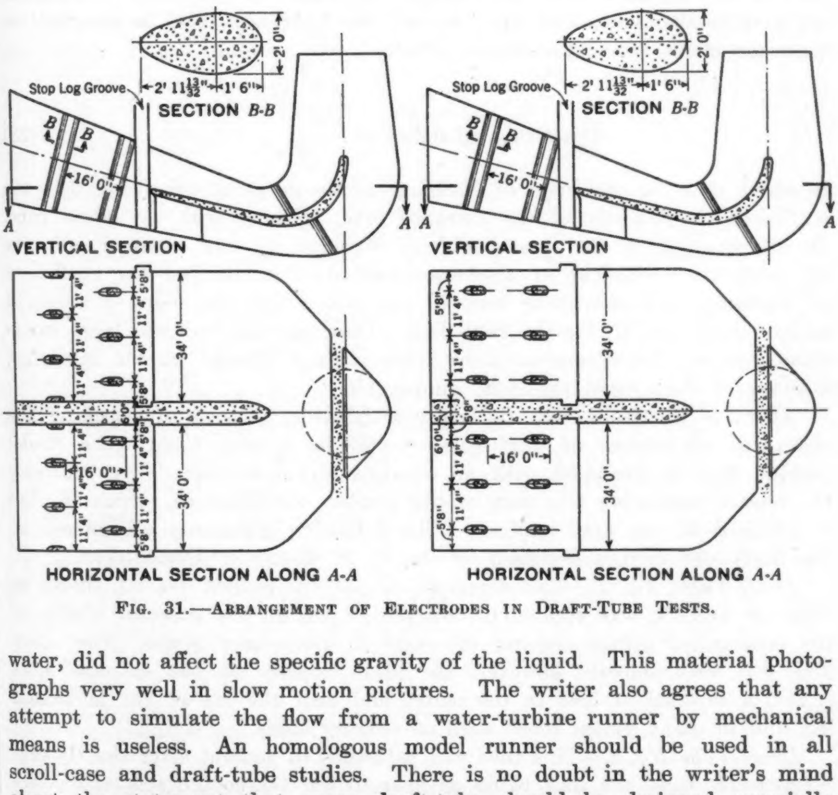

FIG. 31.—ARRANGEMENT OF ELECTRODES IN DRAFT-TUBE TESTS.

water, did not affect the specific gravity of the liquid. This material photographs very well in slow motion pictures. The writer also agrees that any attempt to simulate the flow from a water-turbine runner by mechanical means is useless. An homologous model runner should be used in all scroll-case and draft-tube studies. There is no doubt in the writer's mind about the statement that every draft-tube should be designed especially for its own setting and turbine.

The writer would carry the increasing diminishing degree of the area in the elbow a little further and reduce the sections around the bend, giving the flow a slight acceleration. All model tests showed that when the velocity was slightly accelerated, very smooth flow resulted. This can also be observed by inspecting Bend No. 5, in Fig. 9(*b*) of the paper. After the flow begins to decelerate, it moves in various directions, but while it is accelerating, it is noticeably smooth.

A method of measuring the efficiency of elbows and draft-tubes when the water has a whirling component is very important and such measurement could be accomplished in at least three ways: (1) One with a small tube with two holes in the sides located diametrically opposite and then

placed into the flow with the holes 90° from the line of flow; (2) another method would be to use the outside piezometers and correct for the centrifugal force of the whirling body of water; and (3) the third suggestion would be to use the method developed by the Reclamation Bureau and used in the War Department laboratory at Portland; that is, find the entrance loss in the entrance tube without the draft-tube in place for the various whirling components, and then operate the entrance tube in conjunction with the elbow tube. Combining results:

$$\text{Draft-tube efficiency} = \frac{\left(\frac{C_2}{C_1}\right)^2 - 1}{(C_2)^2} \quad \text{.............. (28)}$$

in which C_1 = the coefficient of discharge of the entrance tube; and C_2 = the coefficient of discharge of the entrance tube, together with the elbow tube. In all experiments the flow should be corrected to one common quantity by using the coefficient of discharge method; that is, find the coefficient of discharge and substitute back in the orifice formula, using a constant rate of discharge, Q, for the final data. This may not indicate large variations because the efficiency varies little with a change in the quantity; however, it does make the data comparable.

There is no doubt that the most satisfactory method of checking the operating advantages of draft-tubes would be to run them on a model turbine that is equipped with an accurate dynamometer. This will give the relative operating efficiency of the various combinations, which is what is required in the final analysis. The following comments are offered on the draft-tube models designed by the U. S. Engineer Department.

Draft-Tube No. 1.—The principle of the fan-shaped jet, as shown in Figs. 28 and 29, was applied to this design within the practical limits of the economical center spacing of units in the power house. The elbow flow was very smooth; however, the high velocity at the extreme sides caused a reversal of flow in the center and near the top of the horizontal leg, and in many places there were no-velocity areas.

Draft-Tube No. 2.—This tube was patterned in general after the Ryberg-Schwerstadt tube, the unit being as near to the Bonneville size and power as any other Kaplan turbine then in operation. As the first tests show, in Table 6, this tube gave fairly good results. The tube was then taken to the U. S. Engineer Laboratory, at Linnton, Ore., and, as a result of slow motion pictures and general observation, several alterations were made: (*a*) The deceleration around the bend was changed to a slight acceleration; (*b*) the top of the vertical leg near the bend was lowered to fill a no-velocity area; and (*c*) the angle of the elbow sides was reduced with respect to the center line. All these changes indicated a marked improvement in the flow conditions and, based on the same method of testing as that done under the direction of Professor Mockmore, showed an average efficiency of 70.0% an increase of about 5 per cent.

The fact that this increase was due to the visual study would indicate that many additional improvements would be possible with added research.

This tube would give very good operating results on a turbine having a small whirling component in either the forward or backward direction. The elbow is too short and the splitter too high in the vertical leg for a wheel having a large whirling component which would set up violent disturbances

TABLE 6.—LABORATORY TEST DATA AFTER ALTERATIONS (*a*), (*b*) AND (*c*) WERE MADE, DRAFT-TUBE NO. 2; $A = 0.1967$ SQUARE FOOT

Run No.	QUANTITY		TANK		Throat gage	$V = \frac{Q}{A}$	$\frac{V^2}{2g}$	Velocity head reclaimed, Column (5) − Column (6)	Percentage efficiency, Column (9) ÷ Column (8)
	Gage	In second-feet	*A*	*B*					
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1	0.691	..	3.300	3.084	2.809	..	..	0.275	..
2	0.693	..	2.851	2.630	2.353	..	..	0.277	..
3	0.694	..	2.457	2.239	1.958	..	..	0.281	..
Average	0.693	0.997	..	..	..	5.07	0.399	0.278	69.7
4	0.671	..	3.276	3.086	2.848	..	..	0.238	..
5	0.671	..	2.860	2.663	2.424	..	..	0.239	..
6	0.672	..	2.460	2.266	2.026	..	..	0.240	..
Average	0.671	0.920	..	..	..	4.67	0.339	0.239	70.5
7	0.629	..	3.289	3.148	2.978	..	..	0.170	..
8	0.629	..	2.854	2.711	2.540	..	..	0.171	..
9	0.630	..	2.423	2.279	2.103	..	..	0.176	..
Average	0.629	0.783	..	..	..	3.99	0.247	0.172	69.6
10*	0.602	..	3.297	3.180	3.043	..	..	0.137	..
11*	0.602	..	2.831	2.712	2.578	..	..	0.134	..
12*	0.601	..	2.466	2.348	2.208	..	..	0.140	..
Average	0.602	0.701	..	..	..	3.56	0.197	0.137	69.5
13*	0.562	..	3.338	3.250	3.151	..	..	0.099	..
14*	0.564	..	2.843	2.751	2.651	..	..	0.100	..
15*	0.564	..	2.427	2.338	2.237	..	..	0.101	..
Average	0.563	0.593	..	..	..	3.01	0.141	0.100	70.8
Average	..	..	..	..	..	..	..	..	70.0

* Data, October 17, 1934.

and reduce the efficiency. All tests definitely showed that wheels giving a large whirling component should have the splitter located above one-third of the way around the board as recommended by Professor Mockmore.

Draft-Tube No. 3.—This tube incorporated the flat jet idea that was used in Tube No. 1, together with the spreading-tube principle as used by some turbine manufacturers. The results were practically identical to those found in Tube No. 1, and the flow conditions within the tube were quite similar.

Draft-Tube No. 4.—This tube was designed and tested by Professor Mockmore. The writer did not see it operate, and, therefore, will make no comment on general operation.

Draft-Tube No. 5.—This draft-tube (not shown in Professor Mockmore's paper) was designed so that the entire whirling component was transferred

into velocity head without the use of splitters; only one vertical splitter was used and that was for structural reasons. The one model that was made showed very good efficiency. No alterations were made, due to the lack of time, but the writer is confident that, with several alterations and re-proportioning, the tube could be made into an excellent draft-tube. The cost of construction should not exceed the cost of splitter tubes.

An interesting phenomenon occurs when under operation; that is, no matter whether the entrance whirling component is 1° or 50°, the flow is evenly divided between the two exit channels with no eddies or high velocity areas. The tail-race is exceedingly smooth under all load conditions. The slow motion pictures showed that this double spiral was completely dissipated before passing around the bend.

TABLE 7.—Laboratory Comparison of Straight-Line and Parabolic Deceleration in Vertical Leg of Draft-Tube No. 2

Vanes	$H_a - H_b$	$\sqrt{H_a - H_b}$	$H_a - H_b$ corrected for Q difference	h_2	$\sqrt{h_2}$	Q	C_2*
(*a*) With Trumpet-Shaped Vertical Leg							
Straight	0.155	0.394	0.159	0.151	0.389	0.771	1.270
	0.250	0.500	0.260	0.247	0.497	0.980	1.275
7½°	0.147	0.3835	0.151	0.143	0.378	0.771	1.307
	0.239	0.489	0.247	0.234	0.484	0.983	1.310
15°	0.181	0.426	0.186	0.178	0.422	0.771	1.170
	0.289	0.538	0.297	0.284	0.553	0.986	1.190
30°	0.366	0.605	0.375	0.376	0.606	0.771	0.815
	0.590	0.768	0.610	0.597	0.772	0.983	0.821
45°	1.488	1.220	1.500	1.492	1.221	0.777	0.405
	2.371	1.540	2.440	2.427	1.560	0.986	0.407
(*b*) With Straight Conical Vertical Leg							
Straight	0.144	0.379	0.150	0.142	0.377	0.764	1.312
	0.234	0.484	0.241	0.228	0.474	0.986	1.337
7½°	0.139	0.373	0.145	0.137	0.370	0.764	1.336
	0.223	0.472	0.231	0.218	0.467	0.983	1.356
15°	0.173	0.416	0.177	0.169	0.411	0.771	1.203
	0.281	0.530	0.291	0.278	0.527	0.983	1.202
30°	0.348	0.590	0.356	0.348	0.590	0.771	0.838
	0.564	0.751	0.584	0.571	0.756	0.983	0.839
45°	1.386	1.178	1.420	1.412	1.189	0.771	0.416
	2.339	1.528	2.370	2.357	1.535	0.992	0.413

* C_2 = coefficient of discharge.

The writer cannot see why Tube No. 4 which contains many of the author's recommendations for designs of draft-tubes, should show a lower efficiency than the Tube No. 2 which is similar in general shape but does not contain any recommended designs. The trumpet-shaped vertical leg or straight-line deceleration showed less efficiency when installed on Tube No. 2 than the straight cone. The data given in Table 7 are offered as evidence.

The writer agrees that a sudden deceleration near the runner may cause cavitation, but he does not believe that the difference between the straight cone and the trumpet-shaped vertical leg would be large enough to cause any trouble. It would be possible, however, to set up eddy and shock losses at the lower end of the trumpet-shaped vertical leg if the deceleration was relatively large because the angle of flare (Angle *A*, Fig. 32) would exceed the angle that water will follow without leaving the surface, where a smaller angle (Angle *B*, Fig. 32), might be satisfactory. The trumpet-shaped vertical leg has one decided advantage (if Angle *A* is not too large) and that is that this change in shape will naturally conform to the widening required within the bend.

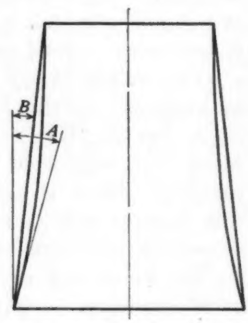

FIG. 32.

Table 8 is submitted as evidence that for a flow that is either straight or with a small whirling component, the location of the splitter as shown on Tube No. 2 is the most efficient. There is a pos-

TABLE 8.—LABORATORY TEST DATA OF SPLITTER ALTERATIONS, DRAFT-TUBE NO. 2

Flow	Basic Q, in cubic feet per second	Measured Q, in cubic feet per second	$H_a - H_b$	$\sqrt{H_a - H_b}$	$H_a - H_b$ corrected for Q difference	h_2 $H_a - H_b$ corrected for Q difference and exit $\frac{V_2}{2g}$	$\sqrt{h_2}$	C_2*
(*a*) SPLITTER IN ORIGINAL POSITION								
Straight	1.000 0.780	0.986 0.764	0.234 0.144	0.484 0.379	0.241 0.150	0.228 0.142	0.474 0.377	1.337 1.312
(*b*) 2-INCH CUT-OFF FROM TOP OF HORIZONTAL SPLITTER								
Straight	1.000 0.780	0.961 0.743	0.226 0.141	0.475 0.375	0.494 0.394	0.244 0.155	0.481 0.383	1.318 1.291
(*c*) 4-INCH CUT-OFF FROM TOP OF HORIZONTAL SPLITTER								
Straight	1.000 0.780	0.958 0.795	0.237 0.174	0.487 0.417	0.508 0.409	0.258 0.167	0.495 0.399	1.281 1.239
(*d*) HORIZONTAL SPLITTER REMOVED								
Straight	1.000 0.780	0.969 0.783	0.295 0.203	0.543 0.451	0.560 0.449	0.314 0.202	0.549 0.440	1.155 1.124
(*e*) WITH FIN ON TOP OF SPLITTER								
Straight	1.000 0.780	0.969 0.761	0.220 0.139	0.469 0.373	0.234 0.146	0.221 0.138	0.470 0.371	1.349 1.334

* C_2 = Coefficient of discharge.

sibility that with the splitter removed the change in rate of deceleration caused the drop in efficiency. The fin on top of the splitter would no doubt be impractical, according to the author, due to the large variation in whirling component; but no doubt it could be used to good advantage in pipe elbows with a fixed angle of whirl.

The writer agrees that the whirling component changes from forward to backward in the Francis type of turbines depending upon load conditions, but for Kaplan propeller types within the normal range of specific speed this component is always forward in the direction of the propeller rotation. There is belief in the writer's mind that one could capitalize on this phenomenon and the energy within this rotating body of water regained in the form of useful work (Draft-Tube No. 5, for example).

The writer can not subscribe to the belief expressed by Professor Mockmore regarding a central rotating core in the vertical leg moving toward the entrance of the tube. This may have been possible when the core was composed mostly of air, but in all tests at the Portland laboratory where a neutral buoyancy material was used to give the flow pattern, no reversal of flow was observed under any operating condition.

Considerable advance in the art of constructing pyralin models has been made in the past few years. One arrangement that gave very accurate results without wrinkling or discoloring the pyralin consisted of an electric oven, thermostatically controlled, into which the moulds for forming the models were placed. The moulds with the pyralin sheets in place were forced together by a hydraulic ram operated from the outside, one side of the oven containing a mica window for observation. After the moulds were pressed together completing the forming operation, the oven was allowed to cool before they were removed. Motion picture cameras taking 64 frames per sec are fast enough to photograph the flow in models of this size. From experience, it is recommended that they be taken at night under artificial light.

The longer draft-tube should be the most efficient for two important reasons: (1) It effects a greater re-gain in velocity head; and (2) it allows whirling and elbow disturbances a longer passage in which to straighten out and make a better seal at the exit of the tube.

The writer would like to make the following suggestion for design of elbow draft-tubes: Carry the circular and elliptical sections down as far as possible in the vertical leg; in order to make the bend, select the inside and outside radii by experience and model testing; reduce the section areas slightly, allowing the top and bottom to conform to the fixed radii and the sides to the correct areas, the flow lines, of course, to be fair at all times. The flow can be decelerated finally in the horizontal leg. In large units install at least two vertical splitters in the horizontal leg and extend them about one-fourth of the way around the bend, one horizontal splitter which does not extend more than one-third of the way around the elbow, and six or eight vertical stream-lined struts in the exit half of the horizontal leg.

The following list gives an idea of many features that could possibly be improved by visual studies of draft-tubes for various specific speeds and types of wheels over a wide range of operating speeds: The location of the splitter; the shape of the splitter; the number of vertical splitters; the proper relation of deceleration and acceleration; shock and eddy losses; no-velocity areas; high velocity areas; reversals of flow; cavitation; vibration; straight-line or parabolic deceleration; minimum exit velocity; use of stream-lined vertical struts; ratio of total depth to diameter of wheel; and inside and outside radii of elbow.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

ECONOMICS OF HIGHWAY-BRIDGE FLOORINGS OF VARIOUS UNIT WEIGHTS

Discussion

BY MESSRS. W. G. FOWLER, AND PHILIP A. FRANKLIN

W. G. FOWLER,[10] ASSOC. M. AM. SOC. C. E. (by letter).[10a]—The problem of designing highway bridge floors has been attracting increasing attention from engineers during recent years, and the importance of this phase of bridge construction is attested by the number and variety of floors introduced during the last decade. To the best of the writer's information, this paper represents the first public attempt to correlate the knowledge relating to the various types of floors and to present it in a rational manner.

The curves and data presented by the author should prove invaluable as a means of making rapid comparisons between various types of floors in a preliminary study. The writer has made some studies of typical bridges with "standard flooring" and the pier heights assumed in the paper, and he finds a close check with the values presented by the author. However, if slightly different assumptions are made for the pier heights, results are obtained which depart rather widely from the costs indicated by the curves of partial unit costs. Because of this fact, the writer feels that these curves could be made much more valuable and easier to understand if the items of superstructure costs were separated from substructure costs. Substructures are usually a special problem for every bridge location, and the writer has never yet been able to set up curves or tables sufficiently comprehensive to be very useful in such studies.

The question of superstructure costs lends itself much more readily to standardization. Many engineers prepare their preliminary studies of a particular bridge structure by making parallel estimates of the superstructure from curves or tables, and of the substructure from special studies adapted to the location; and then they assemble the results in various combinations to determine the most economical arrangement. For this reason, the aforementioned division of data would be very useful.

NOTE.—The paper by J. A. L. Waddell, Hon. M. Am. Soc. C. E., was published in February, 1937, *Proceedings*. Discussion on this paper has appeared in *Proceedings* as follows: April, 1937, by Jonathan Jones, M. Am. Soc. C. E.; and in June, 1937, by Messrs Henry C. Tammen, Miller McClintock, Joseph G. Shryock, and C. Calor Mota.

[10] Office Engr., Black & Veatch, Cons. Engrs., Kansas City, Mo.

[10a] Received by the Secretary August 12, 1937.

Philip A. Franklin,[11] M. Am. Soc. C. E. (by letter).[11a]—It is with economics of first cost that the author has concerned himself and to that phase of the problem he has confined his paper. Very definitely, he states in his "Introduction" that he is "ignoring all claims for superiority * * *," and, in his closing paragraph, warns the discussers of the paper against advocating "special interests." All this is right and proper and the author has done a thorough and complete job of giving the profession a valuable working tool.

The problem of choice of floor still remains and the duty of the engineer is to solve it by balancing the first cost of the various types against their desirability. This involves a knowledge of relative costs as determined by the curves and tables of the paper and also an appreciation of the values of the intangibles not susceptible of reduction to such curves and tables.

Mr. Waddell has simplified the problem by separating the factors into tangibles and intangibles and reducing the tangibles to as efficient a tool for solving first cost as the members of the profession have had presented to them for many years. If the economics of first cost were the ultimate answer to the problem of designing bridges, the consideration of such a paper would be a matter for a convention of purchasing agents and would not be worthy of the efforts of a body of engineers.

The curves and tables are only part of the necessary tools, however, and they must be used in combination with other tools in order to shape the materials entering into the design problem into a product commensurate with the reputation of the engineer. The consideration of this larger problem of the "Ultimate Economics of Highway Bridge Floors" is quite worthy of the efforts of the profession. It should not be hampered by the limitations of mathematics in the discussion, but the merits and shortcomings of all the various types of floors in common use should be reviewed; the factors which lead to desirable features as well as those which produce undesirable results should be listed; and conclusions should be drawn concerning the attributes of the perfect floor.

That there are defects in all the types of floors under discussion is well known to all bridge engineers. The heavier concrete floors are liable to surface abrasion, becoming rough and requiring re-surfacing. It is difficult to keep them from developing cracks due to negative bending, thereby allowing percolation of water to the supporting steel, and causing corrosion in places difficult to inspect and repair. The lighter concrete floors, containing aggregate of lesser weight, relieve the load on the supporting structure, but are still less abrasion-resistant than the heavier types. The filled-grid types are less subject to abrasion, and are still lighter; but they have a continuous metal surface beneath them that presents a maintenance and inspection problem in an inaccessible location. If they are not properly designed they are also likely to crack because of negative bending, with higher repair costs than the plain concrete floors. The inverted trough type of filled floor, if filled with alternate strips of thin and thick filler

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[11] With Bridge Decking Div., Blaw Knox Co., Blawnox, Pa.

[11a] Received by the Secretary August 26, 1937.

covered with a bituminous wearing surface, will become wavy under traffic and will require frequent re-surfacing. If this type is filled with concrete, the concrete and the surfacing will crack along the lines where the depth of the filler changes. It also has the same faults as the other all-metal under-surface floors in that it is more or less inaccessible for inspection and maintenance. The all-metal solid floor, with checkered-plate wearing surface, is also subject to corrosion on the under side and is anything but a safe driving surface in wet or icy weather, or even in summer when the grease-drip oils the surface. The markings soon wear off and only a plain steel plate is left.

All the solid surface types can only have good driving surfaces in fair weather. In snowy, sleety, or even in wet weather, they present a surface to traffic different than in dry weather and much more hazardous.

The open-grid floors require a secondary system of supports and complicate the original detailing and shop work as measured by present-day routine methods. Their greatest drawback is their novelty. For instance, the fact that some engineers "view with alarm" the supposed lack of lateral rigidity in the open floor when no consideration has ever been allowed on that score for any of the solid floors, is quite puzzling. Lateral bracing always has been, and probably always will be, designed for the full effect of some extremely high wind, and the effect of any stiffening due to the floor has been taken as an "added safety factor." The steel in the open-grid floors forms a rather effective horizontal girder of very large moment of inertia, with ample riveting to carry horizontal shear between the various longitudinal members. These, in turn, are welded to their stiffeners in the form of the transverse carrying members and these again to the longitudinal stringers forming a three-member laminated system much more resistant to distortion or to disruption than any concrete floor. To the writer, at least, the idea of putting metal in the lateral system, and in the stiffening truss of a suspension bridge, answers the requirement of "a dollar spent in the main structure is better invested than a dollar spent in local stiffening." It is better than to spend it for more costly floor construction which, formerly, was never credited with lateral stiffness.

Another principal fallacy is that water, in itself, is responsible for corrosion. Corrosion is due to water in combination with other compounds and conditions, and the longer these factors are allowed to remain in combination the faster the corrosion proceeds. Conversely, the speedy breaking up of the combination is the best preventive of corrosion. The sunlight goes through an open grid to the supporting stringers and floor-beams. Each passing vehicle fans the grid and the supports with an intense blast of air. The drip from the engines coats the surface of the grid members. The dirt and dust falling through the grid are dried by the ventilation and forced draft and are blown away so that corrosion does not attack either the supports or the much-feared $\frac{3}{16}$-in. metal of the grid.

Naturally, the consultant is interested primarily in his reputation for conservative design; the fabricator's engineer is interested in securing a

contract by offering a cheaper structure than his competitor while remaining within the letter of the specification. The inventor, whether or not he is in either of the foregoing classes, is interested in having his "brain-child" accepted in "the best circles" and in securing a royalty for it.

This leaves the prospective owner or the unbiased engineer in the position of having to satisfy himself as to the desirability of the various floors offered, and then to choose from them by balancing the costs against the merits. Since only approximately 10% of the membership of the Society is directly interested in bridges and probably not 10% of these are in either of the aforementioned biased classes, it should be apparent that the only hope for any of the other 90% of the members to "sift the wheat from the chaff" is to make their own investigation into the intangibles not covered in the paper.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

FLOOD PROTECTION DATA

PROGRESS REPORT OF THE COMMITTEE

Discussion

By Robert F. Ewald, M. Am. Soc. C. E.

Robert F. Ewald,[10] M. Am. Soc. C. E. (by letter).[10a]—In its progress report the Committtee lists the "human tendency to generalize" as one of the main obstacles to progress in the proper interpretation of flood data. Generalization on that subject prevails because of the lack of fundamental data, but the continued accumulation of information and the development of new lines of approach now give promise of greater accuracy in the solution of the baffling problem of "how large a flood must be provided for."

For many years engineers have been trying to set up a formula for estimating maximum future floods, using equations of the general form,

$$Q = C\,M^{y} \qquad (1)$$

in which Q is the discharge; M is the area of the water-shed (hereinafter termed drainage area); C is a constant for all values of M in the same climatic or topographic regions, but varying as much as 1 000% in passing from one region to another; and y is an empirical exponent of vague significance. Many attempts have been made to derive values of this exponent from the slopes of line-enveloping flood data when plotted on logarithmic paper, but thus far consistent results have not been produced. During recent years there has been a tendency toward assigning the value of 0.5 to the exponent for all sizes of drainage areas probably because certain reasoning, growing out of the use of unit hydrographs, indicates that peak discharges are a function of the time required for the water to travel from the point of precipitation to the principal drainage channel and, with all other factors remaining constant, y then becomes a function of the square root of the drainage area.

Note.—The Progress Report of the Committee on Flood-Protection Data was presented at the Annual Meeting, New York, N. Y., January 20, 1937, and published in March, 1937, *Proceedings*. Discussion on this report has appeared in *Proceedings*, as follows: September, 1937, by Messrs. John C. Hoyt, and H. K. Barrows.

[10] Hydr. Engr., Aluminum Co. of America, Pittsburgh, Pa.

[10a] Received by the Secretary July 30, 1937.

Some of the "other factors," however, are far from being a constant for all sizes of drainage areas, and there is no basis for the assumption that their variations are compensatory to such extent as will permit the use of an average value of 0.5 for the exponent. Many engineers are well aware of the fact that enveloping curves based on a function of $M^{0.5}$ do not satisfactorily fit available flood discharge data. If adjusted closely to the plotted data for drainage areas of less than 1 000 sq miles, the curves lie far above the data for larger areas, and if adjusted to the data for areas larger than 1 000 sq miles they fall below many points for smaller areas. The direct evidence of the plotted data, as well as that derived from certain analyses to be presented later, seems to indicate that not only should the constant, C, be varied to suit differing climatic conditions and topographic regions, but the exponent, y, should also be varied to suit the differing rates and durations of run-off, as affected by the size of the drainage area. The equation then involves so many variables that definite solutions by its use are impractical and its only utility is as a rack upon which may be hung, for classification, the deductions obtained by other more orderly methods.

In studying maximum floods it has no doubt occurred to some engineers that, for physical reasons, there must be a definite limit to the quantity of rain that can fall on a given area in a definite time, and that data could be accumulated from which curves might be developed showing the maximum run-off related to the size of drainage areas for each climatic region. With such curves available, the problem of variations in the other factors such as topography, shape of drainage area, and duration of storm, could be readily and accurately allowed for by solving it with the aid of unit hydrographs. This method of approach would not only eliminate the uncertainties inherent in the use of general equations, which involve uncertain exponents and constants, varying by 1 000% and selected only by judgment, but would also eliminate the necessity for attempting equally uncertain solutions based on this or that probability method. The writer has studied this approach to the problem with rather interesting results and will outline a simple case, more to indicate the importance of certain points involved than to set up a new formulation.

Well defined curves of maximum run-off for various climatic regions are not available, but an approximation to such a curve, applicable to that part of the United States east of the Mississippi River, was prepared in the following manner.

On Fig. 1 is shown that part of the 1-day depth-area curve of rainfall for the great Alabama storm of March, 1929, for drainage areas between 100 and 2 500 sq miles.[11] This curve, as such, is not used in the analysis, but is introduced to indicate how the average storm rainfall, measured in inches, varies with the size of the area. The Alabama curve was selected because it shows greater intensities of rainfall on areas of the sizes under consideration than any storm in the United States east of the Mississippi for which data are now (1937) available. In comparison with hundreds of

[11] Technical Reports, Miami Conservancy Dist., Pt. V, Revised 1936, p. 268.

other storms the rate of rainfall in the Alabama storm seemed to vary with the area in a nearly normal manner. Unfortunately, no method of analysis has been developed by which a reasonably accurate depth-area curve of run-off can be computed from that curve, and data from stream-flow measurements of the floods resulting from that storm are not available in amount sufficient to insure an accurate determination.

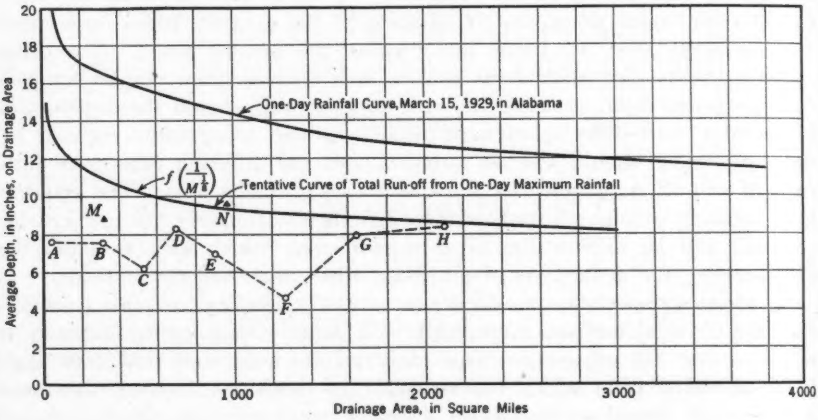

FIG. 1.

Lacking data from the Alabama flood for constructing the run-off curve, the curve in Fig. 1 was based on all the reliable data that the writer has been able to secure with respect to total run-off from unusual storms of approximately 24-hr duration in the United States, east of the Mississippi, for drainage areas of less than 2 500 sq miles. These data are summarized in Table 1. The points representing data for which complete run-off records are available are connected on the exhibit by an irregular line. Two points for the great Carolina storm of 1916 are shown separately because they do not conform strictly to the conditions that the data cover 1-day rainfalls, but they are so near that the error due to the deviation must be small. Complete run-off records for extraordinary flood flows on drainage areas less than 300 sq miles are with one exception not available. Some estimates have been made of the total run-off from drainage areas within that range, based on the peak discharges which, in turn, have been estimated by the slope velocity method. These estimates, however, show so many inconsistencies that the writer has rejected them, with the exception of the possibility that the total run-off on Glen Creek, Watkins Glen, N. Y., with a drainage area of 21 sq miles, during the July, 1935, flood, may have reached 15 in. No point representing this figure is shown because of the uncertainty of the data, nevertheless the indication has been considered in constructing the run-off curve.

There is no physical relationship between the rainfall and run-off curves shown on Fig. 1. The marked difference between the two, each of which

TABLE 1.—TOTAL RUN-OFF FROM STORMS WITH EXCEPTIONALLY HEAVY PRECIPITATION EAST OF THE MISSISSIPPI RIVER, FROM 24-HOUR RAINFALLS

Point on Fig. 1	Stream	Location	Date	DISCHARGE, IN CUBIC FEET PER SECOND		Flood run-off, in inches	Drainage area, in square miles	Authority
				Peak	Ground			
(a) FROM 24-HOUR RAINFALLS								
A	Cane Creek	Bakersville, N. C.	May, 1901	29 000	50 ±	7.3	22	*Engineering News-Record*, August 7, 1902
B	New River	New River, Tenn.	March, 1929	70 000	300 ±	7.5	312	*Bulletin 40* *
C	Ocoee River	Power Station No. 2	April, 1920	47 750	6 000 ±	6.10	530	Tennessee Power Co.
D	White River	West Hartford, Vt.	November, 1927	123 000	1 000 ±	8.30	695	*Water Supply Paper 636*, and correspondence
E	Emery River	Harriman, Tenn.	March, 1929	153 000	1 000 ±	7.25	799	*Bulletin 40* *
F	South Fork Cumberland	Nevelsville, Tenn.	March, 1929	132 000	1 000 ±	4.60	1 260	*Bulletin 40* *
G	Caney Fork	Rock Island, Tenn.	March, 1929	210 000	2 000 ±	7.93	1 640	*Bulletin 40* *
H	Caney Fork	Silver Point, Tenn.	March, 1929	222 000	2 000 ±	8.15	2 100	*Bulletin 40* *
(b) FOR RAINFALLS LASTING SLIGHTLY MORE THAN 24 HOURS								
M	New River, South Fork	Crumpler, N. C.	July, 1916	46 000	500 ±	8.9	325	*Bulletin 31* †
N	French Broad	Asheville, N. C.	July, 1916	110 000	....	9.5 ‡	949	*Bulletin 34* *

* Tennessee Geological Survey in collaboration with U. S. Geological Survey.
† Virginia Geological Survey in collaboration with U. S. Geological Survey.
‡ Estimated from peak discharge and compared with hydrographs of floods of May 22, 1901, December 23, 1918, and January 23, 1906.

is assumed to be the maximum of its kind, is due to several factors which need not be discussed at present. Despite the wide separation of the two curves the writer doubts whether the run-off curve for the Alabama storm would be as high as the one shown, because in no case did the run-off at any of the few gaging stations, for which data for that storm are available, exceed 4 in., even on the smaller areas.

Whether or not the run-off curve is a close approximation to the true maximum for run-off from 1-day rainfalls in the United States east of the Mississippi is admittedly a debatable subject, but the writer believes that all engineers will agree that the conditions represented thereby may be repeated on any drainage area between the Mississippi and the Atlantic Ocean and that the frequency of repetitions of those conditions may be increasing in certain localities at the present time. In any event until the nature of storm movements is better understood, this curve can be considered to present the minimum values of possible maximum run-off from 24-hr storms to be used in estimating flood peaks in the region specified. Having accepted this view, the next step is the estimation of flood peaks therefrom.

The unit-hydrograph method of estimating stream discharges from rainfall is now familiar to most engineers and does not require detailed discussion. The principles governing the use of unit hydrographs lead to the following general statement: The base lengths of hydrographs which portray run-off from a given drainage area from rainfall over equal periods of time will be of equal length, and ordinates at similar points on the time scales will be directly proportional to the total run-off appearing from the rainfall. Therefore, if a complete hydrograph is available for a specified drainage area showing a total run-off of r_0 in. in one day and the peak ordinate is Q_{max}, then the peak ordinate for a run-off of r_0' in. will equal $\left(\frac{r_0'}{r_0}\right) Q_{max}$.

In order to apply the maximum possible run-off method to a specified stream it is necessary to have continuous run-off data for one or more floods of sufficient size to insure that the flood run-off directly accounts for the major part of the precipitation. For the purpose of this discussion, there will be used data for flood discharges in April, 1936, on two contiguous drainage areas in the Smoky Mountain region of North Carolina and Tennessee, having practically identical topography, geology, soil, and flora. The floods followed a 24-hr storm of an intensity to be expected about once in ten years and occurred three days after a lighter storm which had thoroughly soaked the ground but whose surface flow had practically all drained away. The data are summarized in Table 2.

The total run-off, in inches, from Area B was 91% of that on Area A. If the depths of run-off on both areas had been 1.32 in., the flood peak on Area B by direct proportion would have been 13 000 cu ft per sec and the ratio of flood peaks on the two drainage areas due to the storm, $\frac{40\,800}{13\,100}$

= 3.11, or very nearly in the ratio of the square roots of the drainage area, that is, 3.05. This indicates the similarity of conditions on the two areas and tends to confirm the theory that if the total run-offs, in inches, are exactly equal, the peak discharges should be in proportion to the square roots of the drainage areas. A study of the rainfall for this storm shows remarkable uniformity over areas much greater than either Area A or Area B.

TABLE 2.—TYPICAL RUN-OFF DATA FOR 24-HOUR STORM IN CONTIGUOUS MOUNTAINOUS DRAINAGE AREAS

Description	Area A	Area B
Drainage area, in square miles	1 630	175
Discharges, in cubic feet per second:		
Due to ground flow before storm	9 000	2 000
Peak during flood	49 800	13 900
Flood peak due to storm	40 800	11 900
Depth of run-off from storm, in inches	1.32	1.20

Using the data derived from the April, 1936, storms on Areas A and B as a base, the next step is the determination of the peak discharges from those areas for a possible maximum run-off comparable to those which have actually occurred on other drainage areas east of the Mississippi. The tentative curve for total run-off from 1-day maximum rainfall on Fig. 1 shows 8.8 in. for a drainage area of 1 630 sq miles and 11.8 in. for 175 sq miles. By direct proportion the peak discharge for these run-offs would be $\frac{8.8}{1.32} \times 40\,800 = 272\,000$ cu ft per sec for Area A and $\frac{11.8}{1.20} \times 11\,900 = 117\,000$ cu ft per sec for Area B. Adding the ground flow shown in Table 2, the resulting expected flood peaks, based solely on run-off from floods which have actually occurred in recent years, are: 233 000 cu ft per sec for Area A and 98 000 cu ft per sec for Area B.

Continuing the analysis, Fig. 2 shows plottings of the following data:

(1) "Creager's Curve A. For 10 000 year floods."[12]

(2) "Modified Myers Maximum Curve."[13]

(3) Points presenting the peak discharges for the floods listed in Table 1.

(4) Points presenting the peak discharges for certain remarkable floods (see Table 3) in the eastern part of the United States, as given by C. S. Jarvis, M. Am. Soc. C. E.,[14] but not shown in Table 1 because the total run-off is not known.

[12] Hydroelectric Handbook, by William P. Creager and Joel D. Justin, Members, Am. Soc. C. E., p. 54.

[13] *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 994.

[14] "Flood Flow Characteristics," by C. S. Jarvis, Appendix I, Table 2, *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1003.

(5) A curve, *E–E*, Fig. 2 (a function of $M^{0.375}$), through the points representing the possible maximum floods for Areas A and B, calculated in the preceding paragraph.

TABLE 3.—PEAK DISCHARGE FOR EXCEPTIONAL FLOODS IN EASTERN UNITED STATES OF MAGNITUDE COMPARABLE TO THOSE IN TABLE 1, BUT FOR WHICH TOTAL RUN-OFF IS NOT KNOWN

No.	Location	Drainage area, in square miles	Peak discharge, in cubic feet per second
142	Elkhorn Creek, Keystone, W. Va.	44	60 000
175	Chester Creek, near Philadelphia, Pa.	62	62 000
271	Devils Creek, Viele, Iowa	143	86 000

Fig. 2 is almost self-explanatory. Curve *E–E*, derived from run-off data for the greatest known floods of the past forty years east of the Mississippi,

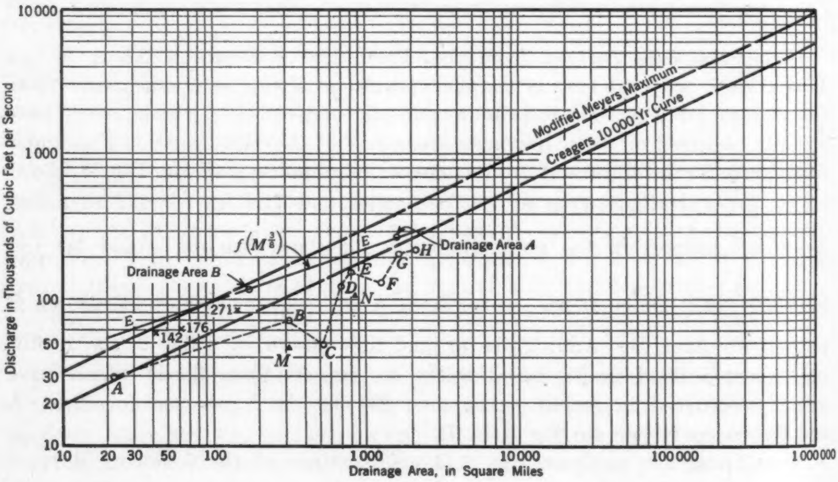

FIG. 2.

and computed with the aid of unit hydrographs for drainage areas with the most rugged and impervious topography east of the Mississippi, agrees remarkably well with peak discharges from floods Nos. 142, 175, and 271, in Mr. Jarvis' tabulation.[14] These floods, which were derived from 24-hr storms, have hitherto been looked upon as freaks and of questionable expectation, but it appears that they are not at all "out of line," and probably were produced by rainfalls approaching the tentative possible maximum corresponding to the run-off curve of Fig. 1. Furthermore, Curve *E–E* is a very close-fitting and all-inclusive envelope for all flood discharges on drainage areas less than 2 000 sq miles when converted to cubic feet per second per square mile and plotted on Mr. Jarvis' graphical

tabulation of flood run-off records.[15] It will not envelop data for certain areas in Texas, Northern Mexico, and possibly India, data for which are not shown in Mr. Jarvis' tabulation, but have been published since the appearance of his paper.

Curve *E–E*, Fig. 2, ostensibly covers only peak discharges for run-off due to 24-hr storms, and the possibilities of greater peaks as the result of more protracted storms were not investigated beyond establishing the fact that on Drainage Area A (1 630 sq miles) combinations of daily run-off hydrographs for a total run-off of 10.75 in. derived from a 5-day storm, and 14 in. derived from a 7-day storm, in no case showed peak discharges as great as those derived for the possible maximum 24-hr storm. The 10.75-in. run-off was established by proportion using data for the Miami (Ohio) flood of 1913. One of the various combinations of daily distributions of rainfall tried was the same as occurred during that storm. The 14-in. run-off was established by proportion, using data for the Obion River, in Western Tennessee, during the great Ohio Valley storm of 1937. One of the various combinations of daily distribution of rainfall tried was the same as that which occurred during that storm. Distributions involving maximum 24-hr rainfall followed by rainfall in amount to complete the totals on subsequent days, were not tried on the theory that there would be little or no moisture remaining in the air capable of precipitation and that, at least, 48 hr would elapse before the next heavy fall could occur.

An interesting speculation now develops: Is it not true that maximum floods on drainage areas of less than 2 000 sq miles are derived only from extraordinary rainfalls occurring in 24 hr, as the result of rapid movement of cold fronts across areas over which the atmosphere is in a saturated condition? Furthermore, does not the tentative run-off curve for maximum rainfall on Fig. 1 show the maximum quantity of water capable of precipitation, that can be held in the atmosphere over a given area up to the time of the passage of a cold front (plus that which can be brought in during the movement of that front) and, therefore, is the maximum that can be precipitated in 24 hr? In an indirect manner this idea gains support from the condition that all the known greatest floods on drainage areas of less than 2 000 sq miles have been derived from rainfalls of relatively short duration. The question can be answered definitely only by an expert meteorologist, but the writer believes that the answer will be forthcoming and will establish definite values from which possible flood peaks may be calculated without resort to speculation inherent in the use of theories of probability.

The physical factors behind the value of 0.375 for the exponent of M in Curve *E–E*, Fig. 2, seem to arise from the following conditions:

(1) All other conditions being equal, peak discharges should be a function of $M^{0.50}$.

(2) From data for widespread storms of maximum intensity it appears that the depth of total run-off varies approximately as $M^{-0.125}$. The run-

[15] *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 994, Pl. IX.

off curve in Fig. 1 when plotted on logarithmic paper, shows a value of — 0.125 for the exponent.

(3) Maximum peak discharges for floods on drainage areas less than 2 000 sq miles, except possibly some areas with very little topographic relief, are the result of 24-hr rainfalls of great intensity, due to movement of cold fronts bringing a relatively small proportion of moisture over areas previously covered by saturated air. (There are some indications that the exponent increases as the size of drainage increases, but the variation for drainage areas of less than 2 000 sq miles is small.)

If the foregoing ideas are correct, the exponent of M should be the algebraic sum of the exponents listed under Conditions (1) and (2), or $0.50 + (-0.125) = 0.375$ for drainage areas as large as 2 000 sq miles. The agreement of "generalized" reasoning and facts leads to the belief that Curve *E–E* will give fairly accurate maximum values for floods in the eastern part of the United States. It may not hold for drainage areas of less than 100 sq miles because of concentration of rainfall on small drainage areas due to wind effects.

The formation of low-pressure troughs with the continuous drawing in of moisture-laden air and the precipitation of that moisture over areas previously cooled, such as in the great Miami flood of 1913, and the Ohio Valley storm of 1937, leading to rainfalls of many days duration, but of less intensity per day, is of secondary importance on drainage areas of less than 2 000 sq miles because of the relative rapidity with which these areas are drained. The effect of storm duration, however, becomes increasingly important as the drainage areas increase in size, apparently compensating completely for the decrease in rate of run-off, so that flood discharge formulas based on a function of $M^{0.50}$ for drainage areas greater than 2 000 sq miles, seem to conform to the facts with reasonable accuracy at least as far as the enveloping curves are concerned.[16]

Considerable data on flood flows in certain parts of Texas[17] published since Mr. Jarvis presented his paper, and some older data from Mexico and India, indicate that peak discharges in those regions materially exceed those indicated by Curve *E–E*, Fig. 2, one point from Texas showing four times as much. These regions apparently are in a class by themselves and subject to a type of storm precipitation not known in the eastern part of the United States. The possibilities indicated by such storms show the great importance of considering each climatic region by itself. The writer notes that mean curves through points for the same storm in the Texas data indicate an exponent of 0.375 for drainage areas less than 2 000 sq miles and 0.50, or even greater, for drainage areas greater than 2 000 sq miles, confirming the fundamental conception with regard to variation in run-off with the size of drainage area discussed herein.

The foregoing discussion is presented with the hope that it may assist in giving direction to further study of flood flow data. The writer believes

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[16] *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 995, Plate IX, in which the equation, $Q = 5\,000$ ($M^{0.50}$) which corresponds very closely to Creager's 10 000-yr curve, obviously forms an excellent envelope for all areas in excess of 2 000 sq miles.

[17] *Engineering News-Record*, November 5, 1936, p. 655.

that it should be possible to determine fairly closely, from a theoretical standpoint and also from stream flow records, the maximum quantity of water which can be precipitated over a given region during a single storm or during a series of storms. Classified as to climatic regions these data can then be used as a base for the application of unit hydrographs for the solution of specific problems. Such hydrographs are now available for many drainage areas and, with the renewed activity in stream-gaging, will probably be available for nearly all drainage areas of any size within a few years. Unfortunately, there is still a marked deficiency in reliable records with respect to total run-off on small drainage areas from near-maximum storms, because such storms are relatively rare and facilities for obtaining complete records on small areas are scattered so widely that there is an element of luck in getting any at all. Would it not be wise to increase, greatly, the number of gaging sections on small drainage areas and to install recording gages at all power or water supply dams where such facilities are not now available, in the expectation that some such installation will eventually supply the much needed data? The most important factor in this respect is to impress engineers with the importance of such data; to put them "on their toes" in the matter of getting the data while the water is "going over the dam" and to release them for general use providing they are reliable and of distinctive importance. Finally, the matter of total run-off from all storm flows should be given greater attention than has been done in the past.

Like many other engineers the writer at one time thought the problem of maximum flood discharges could be solved for some localities by some application of the theory of probability but, after all, rainfall is not at all fortuitous and the frequency of rainfalls of definite quantity varies so greatly from decade to decade and generation to generation as to preclude applications of conventional probability methods with accuracy sufficient for practical purposes, except to indicate the relative frequency of floods in a limited range of values. That procedure may do for estimating the extent to which flood protection should be carried, but it has no part in the design of spillways when failure to provide for the maximum possible flood involves disaster.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

THE PASSAGE OF TURBID WATER THROUGH LAKE MEAD

Discussion

By Messrs. D. M. Forester, A. D. Lewis, G. C. Dobson, and William W. Rubey

D. M. Forester,[29] M. Am. Soc. C. E. (by letter).[29a]—The passage of turbid waters through reservoirs is a phenomenon which has seldom been considered and, in fact, generally not known. Quite often it is assumed that the discharge of turbid water or silt through the discharge gates has been the result of current changes and slides within the reservoir. The flow of turbid water with its apparent resistance to mixing is a common occurrence in settling or clarification basins of water treatment plants. During the early stages of the filling of Lake Mead, when light conditions were satisfactory, the writer could distinctly see the flow of turbid water along the old river channel when observed from an altitude of 300 or 400 ft above the stream bed.

The records of the water treatment plant at Boulder City, Nev., with its intake approximately 1 000 ft down stream from the discharge portals, very closely confirm the observation at the Willow Beach gaging station. These records do not, however, indicate a noticeable variation in the total hardness content of the water during the periods of high turbidity. Analysis of Colorado River water since February, 1935, have recently become available for public inspection in the offices of the U. S. Geological Survey.[30] These analyses show wide variation between the inflow to Lake Mead and the outflow from Boulder Dam. During the first year of storage in Lake Mead the range of the total dissolved solids at the Willow Beach gaging station was 57% of that at Grand Canyon. During the second year the range was 39 per cent. The first-year range at the Grand Canyon gaging

Note.—The paper by Nathan G. Grover, M. Am. Soc. C. E., and Charles L. Howard, Esq., was published in April, 1937, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: June, 1937, by Messrs. O. A. Faris, Paul A. Jones, Carl E. Scofield, and Ivan E. Houk; and September, 1937, by Messrs. William P. Creager, Harold K. Palmer, Morrough P. O'Brien, John C. Page, John H. Bliss, and B. H. Monish.

[29] With U. S. Bureau of Reclamation, Yuma, Ariz.

[29a] Received by the Secretary August 13, 1937.

[30] *Western Construction News*, Vol. XII, No. 6, June, 1937, p. 220.

station was from 242 ppm to 1 332 ppm and, at Willow Beach, from 276 ppm to 897 ppm. During the second year, the range at Grand Canyon was from 258 ppm to 1 215 ppm and at Willow Beach from 393 ppm to 762 ppm. The gate in the tunnel was closed on May 1, 1936, three months after the beginning of the second year.

The dissolved solids and the suspended solids entering and flowing from the lake appear to vary in somewhat similar manner and both indicate that the mixing action between the inflowing water and the water within the reservoir is not pronounced over a short time period.

A statement by O. A. Faris, M. Am. Soc. C. E.,[31] that the flow of silt along the reservoir bottom is due to its having a greater specific gravity than the main body of water in the reservoir, would appear to be conclusive. However, the discharge of turbid water does not occur at all times. At Lake Mead, from March 1 to October 31, 1935, the inflow was never less than 0.11% by weight (1 100 ppm) of suspended solids, the maximum being 5.09% (50 900 ppm) and averaging 0.91% (9 100 ppm). The outflow suspended solids content varied from 2.16% to 0.002%, averaging 0.17%, indicating a deposition of 0.74 per cent. At periods of maximum inflow of suspended solids and corresponding maximum outflow, the indicated deposition was much higher.

From present available information it would appear that the flow of turbid water through a long reservoir is dependent upon the quantity of suspended solids and their particle size. The degree of concentration would directly affect the specific gravity of turbid water. The flow would also be affected, to a lesser degree, by temperature conditions and perhaps by dissolved salts.

Comparison of deposition of the various particle sizes was not given by the authors; however, it is reasonable to assume that deposition of suspended solids would be in proportion to their size.

Any steps which may ultimately lead to the maintenance and conservation of original reservoir capacity cannot be otherwise than valuable. It is the writer's opinion that other economic values, especially in irrigation and municipal supplies, usually would outweigh the advantages secured by increasing the silt discharge through the normal, or low-level outlets.

A. D. Lewis,[32] Esq. (by letter).[32a]—Several features treated in this paper have been described by the writer in connection with similar occurrences in South Africa.[33] It should be noted that for several years past the standard practice at Lake Arthur Reservoir has been to remove a portion of the incoming silt through the valves during the flood by a process which is termed "bottom-flow tapping." There cannot be the least doubt that in certain circumstances there is a flow of flocculated silt along the bottom

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[31] *Transactions*, Am. Soc. C. E., Vol. 101 (1936), p. 251.

[32] Director of Irrig., Union of South Africa, Pretoria, Union of South Africa.

[32a] Received by the Secretary August 17, 1937.

[33] *Communication No. 5* to the Second Congress on Large Dams, Washington, D. C., 1936, entitled "Silting of Four Large Reservoirs in South Africa."

of the reservoir under the comparatively clean water. Observations as to the conditions of the actual flow along the bottom at different points along its course are exceedingly difficult, especially in the case of reservoirs in South Africa which are fed by very flashy floods in intermittent rivers. As far as local observations have gone, it appears that in small floods the depth of the flow of concentrated silt along the bottom is seldom greater than 3 ft within 0.75 mile from the dam walls, and its velocity is probably less than 0.5 ft per sec. The analysis of the silt that reached and passed through the valves given in *Communication No. 5*[33] showed that most of it was flocculated clay. After deflocculation, 78% consisted of particles of smaller diameter than 5 microns, and 83% of these particles were of smaller diameter than 2 microns.

The first conclusion one is inclined to draw is that bottom flow is essentially associated with flocculated clay. The experiments and observations in *Communication No. 5,* suggest some reasons why bottom flow should be possible with flocculated clay in certain states of concentration. When the flocculated clay occurs in concentrations greater than a lower limit which lies between 1.5% and 5% and less than an upper limit which is about 25%, it appears to form with the water enclosed a separate mobile liquid with a defined boundary. It is possible that in concentrations greater than the lower limit the flocculated particles are so close together that they are capable of some cohesion, which increases as the concentration increases and makes the escape of the enclosed water more difficult and the settlement of the particles in still water slower. While the flocculated mass is between these approximate limits of concentration it has a mobility which permits its flow as a separate liquid with a sharp boundary surface. The conditions of flow would naturally be governed by such factors as the concentration and temperature of the liquid and the slope and shape of the bottom of the reservoir.

Flow becomes very sluggish and suddenly almost ceases near the upper limit of concentration, which the writer has termed the "critical point," and the concentration at this critical point is generally about 19 lb per cu ft. At this point the process of settlement might be said to give way to a process of compaction. In concentrations less than about 1.5%, the top particles settle at a rate of about 2 ft per hr without impeding each other, and they pile up on the bottom into a mass which is probably not very mobile and, therefore, not capable of producing bottom flow. In concentrations between 1.5% and 5%, a sharp surface forms at a position which is farther from the bottom as the concentration increases. In concentrations greater than 5%, the sharp surface forms at the top and no independent settlement of particles is possible even at the top; the silt settles as a whole with a sharp upper surface. If the liquid entering the reservoir, and having a concentration of flocculated silt greater than the lower limit, can glide smoothly under the clear water already in the reservoir without dispersion or mixture with the clean water (which would reduce the concentration to less than 1.5%), then it is possible to picture the mobile liquid flowing along the bottom of the reservoir with just

sufficient turbulence to prevent settlement, and a concentration beyond the "critical point" of about 25%, until it is checked at the dam, if no great discharge is taking place through the outlets of the dam. There, in the stiller water, settlement will occur, and the time of settlement to the "critical point" is represented by the formula:

$$t = \frac{c\,L}{15} + 1.2 \qquad (10)$$

in which t is the time of settlement, in hours, to the "critical point"; c is the initial concentration, expressed as pounds of dry silt in 1 cu ft of silty water; and L is the original length of the silty column, in inches.

If the silty concentration is not drawn off within this time, the mass will lose its mobility and will not flow through the outlets. A certain degree of concentration by settlement at the dam wall is necessary; otherwise, an unnecessary quantity of water will be wasted in getting rid of the the silt. In the actual practice of bottom-flow tapping at Lake Arthur Reservoir (although, actually, a maximum of 25% concentration has been obtained in the liquid extracted), in endeavoring to secure the maximum extraction of silt with the minimum wastage of water by regulating the opening of the bottom valves, it has seldom been possible to achieve a greater concentration than 15% over a long period. If, therefore, the water entered the reservoir with a concentration of 5% of flocculated clay, it would be necessary to waste about one-third of it in order to get rid of the silt that reaches the wall by means of bottom-flow tapping; but it should be remembered that, even if this silt had remained in the dam, it would have entrapped a considerable volume of water which could never be extracted as clear water through the valves.

With the foregoing picture of the process of bottom-flow tapping in mind, it is interesting to examine Table 1 and Fig. 2 of the paper. In no case was there any bottom-flow discharge when the percentage of suspended matter entering the reservoir was less than 1.7 per cent. There was no such discharge during the four months of high flow from May to August; all the discharges were during small floods in the remainder of the year. This might be due to the fact that the saline content of the water entering the reservoir was sufficient to cause flocculation during the latter periods, but not sufficient during the former period, as would appear to be indicated by Fig. 2(*a*).

If, however, there was flocculation during the former period, the concentration was usually less than 1.5% and, therefore, too low to permit bottom flow. According to Table 1, if the last six days of August, which had their effect in September, are excluded, on only eight days in the four summer months did the concentration exceed 1.5 per cent. The highest concentration during this period was on June 2, namely, 1.90%, and as the flood at that time was about 50 000 cu ft per sec, it is possible that even if the clay were flocculated, the turbulence on entering the reservoir would have been sufficient to reduce the concentration to less than 1.5 per cent.

It is assumed that throughout 1935 all the discharges occurred through outlets very low in the dam, which would permit extracting the silt below the level at which concentration would occur in still water above the dam. It is understood that, at a later date, all discharges were at a much higher level, and if the foregoing "picture" is correct, it is to be expected that the bottom-flow discharge will not again take place until the reservoir has been filled with settled silt nearly to the level of the outlets that are in use.

In view of the nuisance that such silt may be when the water is drawn off for domestic purposes or for driving turbines, a practical consideration is that it is desirable to have low-level valves that can be opened sufficiently to discharge at a concentration of about 15% when bottom flow is taking place, and, secondly, that when clear water is required from the reservoir for domestic or other purposes reliance should not be placed solely on valves at low levels which might tap the bottom flow, but a series of valves should be provided at such levels that clear water can be tapped at higher levels during periods of bottom flow.

G. C. Dobson,[34] M. Am. Soc. C. E. (by letter).[34a]—After reading this interesting paper, it occurred to the writer that an examination of the many reservoir silting surveys made by the Section of Sedimentation Studies, Division of Research, United States Soil Conservation Service, should throw considerable light on the subject of the passage of silt-laden water through reservoirs. It would seem that, if this silt-laden water passed through the reservoir as an underflow or density current along the bottom, the distribution of silt within the reservoir would be entirely different than if it was diffused generally with the water already stored in the reservoir. If this phenomenon of underflow appeared only rarely, under certain special conditions, but occurred where these special conditions involved very heavy silt loads, the arrangement of silt in the bottom of the reservoir should indicate this condition also.

Fifty-five detailed reservoir surveys, distributed fairly well throughout the United States, have been made in the three years, 1935, 1936, and 1937. The objectives of these surveys did not include the study of underflows or density currents so that any evidence found on this subject is incidental. A thorough analysis of this mass of data and a gathering of exhibits to prove certain theories was not the object of this examination, the intention being rather to review these data and, in a general way, to reveal what they indicate as to the existence of this phenomenon not only in the Southwest, but in other sections of the United States. Several of these reservoirs are too small to be of any benefit in this study. Some others are of such pronounced "channel" type that they are of little value in studying this distribution of silt. However, there are a sufficient number left to give a fair picture of the conditions throughout the country.

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[34] Acting Head, Sedimentation Studies, Div. of Research, Soil Conservation Service, U. S. Dept. of Agriculture, Washington, D. C.

[34a] Received by the Secretary September 1, 1937.

Sufficient time has not elapsed since the impounding of Lake Mead to warrant a silting survey, although a complete original-capacity, contour survey has been made for this purpose by the Soil Conservation Service. A detailed survey of Elephant Butte Reservoir was made in 1935 in co-operation with the U. S. Bureau of Reclamation. This reservoir has been noted by a number of writers as the outstanding example of underflow currents in the United States.

In studying the profile of Elephant Butte Reservoir, showing the maximum depth of silt above the valley floor, it is noted that because of the great fluctuations in the elevation of water surface and the consequent flooding and drying of the area above the narrows, little can be learned from this section of the profile. Thus about 28 miles of the upper part of the profile is eliminated, leaving about 15 miles from the lower end of the narrows, which are about five miles long, to the dam where the distribution of silt should show the effects of density currents if they exist. At the lower end of the narrows the maximum depth of silt on the valley floor is 30 ft. This depth decreases to 12 ft at a point 7 miles down stream where it begins to increase somewhat irregularly until it reaches a depth of 20 ft just above the dam.

The most noticeable point, however, in the arrangement of silt in this reservoir is shown on the cross-sections taken between the dam and the narrows. Regardless of the depth of the silt, or the width or shape of the cross-section, the upper surface of the silt is practically level.

Pausing for a moment to consider what one would expect to find in the distribution of silt, it would seem that, if the silt-laden water flowing into a reservoir entered into general diffusion with the water already stored, a section across the reservoir should show a distribution of velocity which, at least, would be somewhat similar to that within a section taken across the river above its entrance to the reservoir. The area of the reservoir section would be so large compared to the area of the river section that the resulting average velocity would be very low, and it would be difficult even to surmise whether the difference between maximum and average velocities would be more or less pronounced than within the river section. Furthermore, it would seem logical to suppose that the part of the reservoir section formed by the submerged river channel would affect the distribution of these velocities, but that this effect would become less and less as the reservoir sections became larger. In the case of a meandering stream channel in a submerged alluvial bottom, where the channel formed only a small percentage of the total reservoir cross-section, the influence of the channel would be lost, and the slow currents moving down the reservoir would tend to take the shortest route, disregarding the meanderings of the channel. If this is true, the depth of silt within a cross-section would be affected considerably by the depth of water although not necessarily directly proportionally to it; probably it would be more nearly proportional to some power of the depth, slightly greater than one. There would be a tendency for the low places, particularly the channel, to fill faster so

that the upper surface of the silt would tend to approach a level line in cross-section.

If the silt-laden water were to pass through the reservoir as an underflow, it would seem very probable, because of the greater density, that at any section within the reservoirs this underflow would fill the low places, and its upper edge, if it were clearly defined, would be level from bank to bank. If, as supposed, these underflows occur only at flood time, they would be of sufficient volume to overflow the submerged channel, but the cross-sectional area of the underflow would form only a small part of the area of the total cross-section. The influence of the submerged channel on this flow would be much more marked than on a diffused flow so that, it seems probable, the maximum velocity of this underflow would follow the submerged stream channel even in its meanderings in a submerged alluvial valley. Deposition of sediment from an underflow would undoubtedly occur more or less continuously in its flow down the reservoir and, on a cross-section, the depth of this deposition should bear about the same relation to the depth of the underflow that it would to the total depth of the reservoir in diffused flow. Here, again, there would be a tendency to fill the low places faster, causing the upper surface of the silt to approach a level line. The point to be noted, however, is that, in this latter case, the tendency would be much stronger than in the former and it would seem probable that in a comparatively short time underflow conditions would produce a level floor of silt in the reservoir, while in the case of a diffused flow, the tendency would be so slow that it might not reach this condition until the reservoir was practically full of silt.

Because of the great difference in the effective velocities in these two types of flow one would expect to find a marked difference in the longitudinal distribution of the silt. The low velocities of diffused flow should produce heavy deposits in the upper part of the reservoir gradually decreasing in depth, possibly extending all the way to the dam. The pattern would depend, of course, on many factors, such as the length of travel and general shape of the reservoir. The higher velocities of density flow would be expected to carry the silt much farther into the reservoir on an average and probably all the way to the dam, it might even form the deepest deposits immediately above the dam.

Since the level floor of silt was the most striking feature found in the survey of Elephant Butte Reservoir, this condition was looked for in other reservoirs. In only two other cases was it found to a pronounced degree. These examples were the San Carlos Reservoir, on Gila River, in Arizona, and the Gibraltar Reservoir, near Santa Barbara, Calif. Another striking feature was noticed in the profile of the Gibraltar Reservoir. This reservoir is only about four miles long. The upper half of the profile is very irregular for reasons somewhat similar to those described for Elephant Butte, but in the lower 7 000 ft, the depth of silt increases down stream from 27 ft to 85 ft at the dam. The gradient of the channel in this reservoir is very steep which probably accounts for this rapid increase in silt toward the dam. The lower 7 000 ft of this reservoir seems to be rapidly approaching

a level floor of silt longitudinally as well as transversely. The down-stream slope of the silt surface is only about 0.00076 whereas the effective slope of the original channel is about 0.0062.

Aside from these three reservoirs, the surveys show no marked cases of level silt surfaces. At the other extreme is Black Canyon Reservoir, on Payette River, near Emmett, Idaho, where the cross-sections show the deepest deposits of silt at the points of greatest depth, and where these depths of silt are only slightly greater than proportional to the total depth of water. This condition is shown generally throughout the reservoir. In profile, heavy deposits are found in the upper parts of the reservoir as would naturally be expected of a stream carrying sand bed load. These delta deposits extend a considerable distance down into the reservoir. From the lower end of the delta to the dam, there is a gradual lessening in the average depth of the silt.

A condition prevailing in many reservoirs, particularly in those of the Piedmont Region in the Southeast, is a distribution of silt indicating in general a diffusion of the silt-laden water with the water already stored in the reservoir, but showing evidence also that at times there have been underflows. It must be admitted that this evidence is not very pronounced and might be due to other causes. In many of these reservoirs, without regard to length, or the varying shapes of the cross-sections, the profiles, when taken along the thalweg or lowest points of the submerged channel, show a surprisingly constant depth of silt. This is generally true regardless of the meanders of the channel in a submerged alluvial plain. On the cross-sections the depth of silt within the submerged channels seems to be considerably greater than one would expect as a result of settlement from diffused flow. Outside the channel, the distribution of silt seems to be nearly proportional to the depth of water in the reservoir, but there are some places where deposition on the banks of the submerged channel is greater than at other points on the valley floor, indicating that there is some tendency of the silt to deposit in formations similar to natural levees characteristic of the banks of alluvial streams. In a few cases, this heavier deposition was found on what appeared to be a natural levee in the original bottom of the reservoir. One explanation of this might be that for some unknown reason the velocity of the underflow at these points was greatly decreased, resulting in an enlargement of its cross-section and the overflowing of the submerged channel with a still greater reduction in the lateral velocities causing rapid deposition on the near banks. However, as these apparent natural levees of silt are not generally prevalent and are not very pronounced, their value as evidence is questionable.

Another very noticeable feature in these reservoirs is that a negative gradient in the profile of the original channel, as when emerging from a deep pool, is parallel to the silt surface, which shows as great a depth at the high point as at the low. If the distribution of silt in a reservoir was greatly affected by mass flow of the silt itself, it would be expected that the pools in the original channel would be filled and the high points would

show less silt. Further evidence bearing on this question of mass flow of the silt is to be found on the cross-sections of the great majority of these reservoirs where almost uniform layers of silt of considerable depth exist on slopes many times steeper than the gradient of the river or the floor of the valley.

The phenomenon discussed in this paper is too little understood by the Engineering Profession. The authors have rendered a worthy service in presenting the subject in such a thought-provoking manner.

William W. Rubey,[35] Esq. (by letter).[35a]—The flow of muddy or saline or cold water through lakes is of interest to all who are concerned with the natural processes of streams. The formation, in 1937, of a committee of the National Research Council for the study of density currents is evidence of the wide interest in this subject among engineers, physicists, and geologists. To geologists, bottom currents through bodies of standing water are important because, if such currents are common to-day, they must have influenced materially the character and distribution of sediment deposited on (or swept from) the beds of lakes, estuaries, and seas in the past. The best means of appraising the importance of these currents in the past is a knowledge of the conditions that accompany their formation to-day. This paper with its data on muddy flows through Lake Mead is thus a very welcome contribution.

From what is known about the conditions that determine whether fluids of different density will mix turbulently or flow separately, it appears that, for simple homogeneous fluids, the controlling factors are the differences in density and velocity of the two fluids,[36] and that, for complex gradational fluids, the controls are the density gradients and velocity gradients in transitional layers.[37]

The available data for Lake Mead reveal nothing whatever about the thickness of a possible transitional layer or layers between lake water and muddy current. Studies in progress by the U. S. Bureau of Reclamation at Lake Mead, and by others elsewhere, should eventually furnish the required information on this point; but until this information has been assembled, it seems permissible to consider the lake water and the muddy current as two essentially homogeneous fluids, only one of which has an appreciable velocity. Thus, when the density difference between them is great enough and the current velocity low enough, the two fluids are separated by a surface on which waves move down stream. Increasing the current velocity or decreasing the difference in density breaks up these waves and causes mixing.

By this concept, the boundary between lake water and muddy current is determined by the same principles that govern wind waves at a water

[35] Geologist, U. S. Geological Survey, Washington, D. C.

[35a] Received by the Secretary September 13, 1937.

[36] "Handbuch der Ozeanographie," by O. Krümmel, Vol. 2, p. 62, 1911.

[37] "On the Stability of Superposed Streams of Fluids of Different Densities," by S. Goldstein, *Proceedings*, Royal Soc., London, Vol. 132, Ser. A, pp. 524–525, 1931.

surface[38] and current ripples on a sandy stream bed.[39] At high winds the tops of the waves break into fine spray; at high current velocities the sand ripples are swept away and the sand is picked up and carried along, mixed with the stream water. Just as the differences of density between muddy currents and lake water are much less than the differences of density between water and air or between sand and water, so the critical velocities that cause instability and mixing of the muddy currents should be much less than the critical velocities that make wind waves or sand ripples unstable.

The available data permit no close estimate of velocities of the muddy currents that moved through Lake Mead in 1935, but published and unpublished records of the U. S. Geological Survey and the Bureau of Reclamation afford a basis for computing approximate densities of the inflowing and outflowing waters. Inasmuch as water densities were almost certainly one of the fundamental factors that controlled the existence of muddy currents that year, these data appear worth recording in connection with this paper.

The principal variations in density or specific gravity of the river water were those caused by variations in the content of suspended matter. Values of density at different concentrations of suspended matter, calculated on the basis of 0.996 gram per cu cm for pure water (at 84.2° F) and 2.6 grams per cu cm for suspended solids,[40] agree fairly well with the authors' observed values (see Table 2), giving a maximum disagreement of 0.0022 gram per cu cm, and an average of about 0.0010 gram per cu cm. (These disagreements are probably due, in part at least, to differences in salinity.) Assuming these densities for water and solids, 1% of suspended matter should make a density increase of 0.0062 gram per cu cm, 3%, an increase of 0.0188 gram per cu cm, and 5%, an increase of 0.0217 gram per cu cm.

Two other relatively minor causes of variation in density of the river water need to be considered—salinity and temperature. The effects of salinity were estimated (see heading in paper, "Dissolved Matter") on the assumptions that daily sulfate determinations are approximately proportional to total dissolved matter and that total salinity affects density about the same as if it were all sodium sulfate (0.0011 gram per cu cm for 1 000 ppm). The resulting estimates of density are believed to be in error by not more than 0.0006 and commonly about 0.0003 gram per cu cm.

Observations on temperature of the river water at Grand Canyon Gaging Station began April 20, 1936, and unpublished records are available from that date through July 31, 1937. For the period, April 20 to July 31, for which there is a 2-yr record, the 1936 and 1937 records show closely comparable water temperatures day by day, and it seems fair,

[38] "The Physics of Solids and Fluids," Section by L. Prandtl, pp. 225, 266, London, 1930; also, "Physics of the Earth, V Oceanography," by N. H. Heck, M. Am. Soc. C. E., and others, *Bulletin*, National Research Council, pp. 207, 212, 1932.

[39] "On Ripples and Related Sedimentary Surface Forms," by W. H. Bucher, *American Journal of Science*, Vol. 57, 4th Ser., pp. 165, 178–179, 199–207, 1919; also "Treatise on Sedimentation," by W. H. Twenhofel and others, pp. 458–460, 1926.

[40] "Denudation," by R. B. Dole and Herman Stabler, M. Am. Soc. C. E., *Water Supply Paper 234*, U. S. Geological Survey, p. 80, 1909.

therefore, to assume that water temperatures changed in about the same manner in 1935. The detailed records show fair agreement with uniform changes from 43° to 75° between March 1 and July 1, from 75° to 81° between July 1 and August 1, from 81° to 75° between August 1 and September 1, and from 75° to 52° between September 1 and October 31. Densities estimated from uniform rates of temperature change between these respective dates show a maximum of 0.0006 and a mean departure of about 0.0002 gram per cu cm, from densities estimated from the detailed temperature records. For the Willow Beach Gaging Station below Lake Mead, the water temperatures used were those of the water discharged from the lake during the same period.[41]

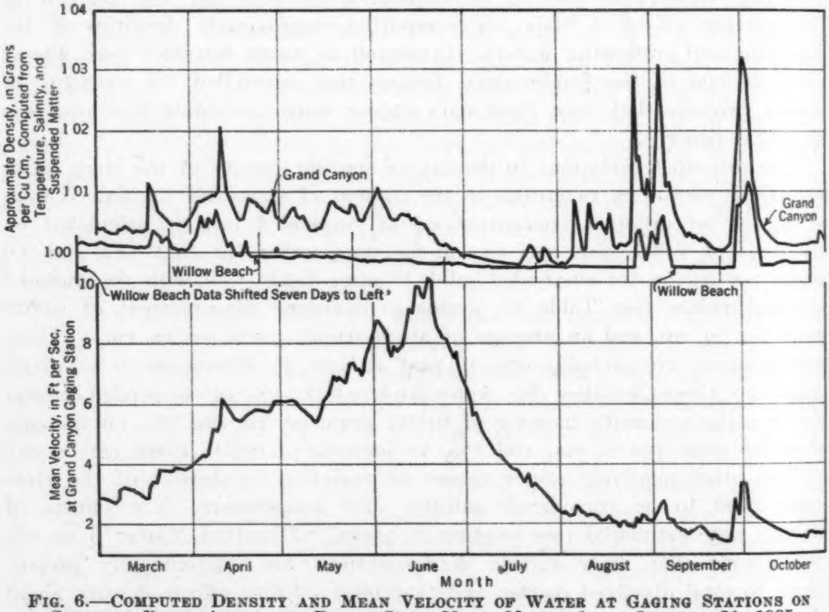

FIG. 6.—COMPUTED DENSITY AND MEAN VELOCITY OF WATER AT GAGING STATIONS ON COLORADO RIVER ABOVE AND BELOW LAKE MEAD, MARCH 1, TO OCTOBER 31, 1935

By combining the effects of the three variables (suspended matter, salinity, and temperature), the daily densities of the water at the two gaging stations above and below Lake Mead were estimated for the period, March 1 to October 31, 1935. These calculated values of density are believed to be in error by not more than about 0.002 and commonly about 0.001 gram per cu cm.

Fig. 6 shows clearly that the density of the river water at the upstream (Grand Canyon) station was not the sole factor that caused muddy currents through Lake Mead. Likewise, if the water discharged from the lake between periods of muddy water is taken as representative of the lake water, the difference in density between river water and lake water was

[41] *Proceedings*, Am. Soc. C. E., June, 1937, Fig. 3, p. 1212.

not the sole factor either. It is true that densities greater than 1.011 grams per cu cm at Grand Canyon were always accompanied by muddy discharges through the reservoir. However, from the duration of individual muddy flows, it is clear that, at times, densities of 1.005 to 1.008 grams per cu cm (and density differences of 0.004 to 0.010) were sufficient, whereas on June 2, for example, a calculated density of 1.0104 grams per cu cm and a density difference of 0.012 grams per cu cm were not sufficient to cause muddy flows.

Of course, these densities of the rapidly flowing water at Grand Canyon Station were not identical with the densities of the same water in the slower current through the reservoir down stream. Important differences are suggested by the fact that the muddy discharges at Willow Beach Station contained considerably less suspended matter than was carried past Grand Canyon Station. This suggestion is confirmed by the fact that the suspended matter in muddy discharges at Willow Beach consisted chiefly of particles less than 20 microns in diameter (see heading, "Sizes of Particles"), whereas this finer sediment at times was only a small part of the total suspended matter at Grand Canyon Station up stream. Evidently, only the finer particles brought in by the river could be kept in suspension by the slower current through the reservoir, and the coarser sediment was dropped somewhere along the way.

Accordingly, the densities of the muddy water after it reached the reservoir were estimated on the assumption that all suspended particles at Grand Canyon larger than 20 microns in diameter (Fig. 2) settled out in the slower current below. These estimated densities show no closer relationship to the existence of muddy flows than do densities calculated from total suspended matter at Grand Canyon. Density differences of less than 0.002 gram per cu cm (estimated from the finer material only) resulted in muddy flows in March and April, but differences of 0.003 to 0.004 gram per cu cm did not cause muddy flows in June and August.

The authors carefully show that the sizes of suspended matter at Grand Canyon were determined in the laboratory from carefully dispersed samples, whereas in the natural river water the finer sediment was partly flocculated into larger masses which settle more rapidly. Inasmuch as the salinity of the river water changed during this period, the degree of flocculation presumably changed also. Thus, the finer material brought into the reservoir in June and August may have been flocculated into larger masses that were dropped by the slower current, with the coarser sediment. If so, the density of the muddy water, after it reached the reservoir, cannot be estimated from the quantity and size of sediment at Grand Canyon without data on variations in the degree of flocculation also.

Whatever may have been the effect of variations in the degree of flocculation, variations in the velocity of the muddy currents themselves were presumably a contributing factor in the formation of currents through the reservoir, for the velocity of these currents would be expected to vary somewhat depending on the density and stage of the incoming water.

However, reliable estimates of rate of flow through the reservoir are difficult to obtain from existing data. The earlier muddy flows of 1935 required about 5 to 7 days, and the later flows about 7 to 9 days, to travel from Grand Canyon to Willow Beach. During this time the length of the lake increased many miles, and variations in velocity through it cannot be estimated with the degree of accuracy required to test their effect.

Lacking reliable data on the velocities of currents through the lake, it seemed that the velocities of incoming water up stream might possibly be a factor in the formation of the muddy currents. Accordingly, the mean velocities at Grand Canyon Station were read from daily gage-height records. These mean velocities (see Fig. 6) are probably fairly accurate because, during the period February 26 to November 3, 1935, the cross-section and mean velocity of the stream at Grand Canyon Station were measured on 117 days (from unpublished records of the U. S. Geological Survey).

On comparing water densities at the two stations with mean velocities at Grand Canyon Station (Fig. 6), it is evident that low or intermediate velocities above the lake accompanied flows of muddy water through it and that, in general, high velocities accompanied the absence of such flows during May and June. It is equally evident, however, that the mean velocities and densities at Grand Canyon Station afford no satisfactory criterion for the existence of the muddy currents. The period of low velocities and high densities, August 6 to 18, was not followed by muddy discharge through the reservoir, although velocities were then lower than, and densities were as great as, those that caused muddy flows in March and April.

The evidence presented herein is largely negative. The available data show no well-defined relationship between the existence of muddy currents through Lake Mead, on the one hand, and the density, density difference, or density and velocity at Grand Canyon, on the other. It seems clear that, in future investigations of the conditions that accompany muddy currents through a lake, direct observations on the degree of flocculation of sediment in the incoming water, and on the velocity or the slope of the muddy currents themselves, will also be desirable.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

NATIONAL ASPECTS OF FLOOD CONTROL

A SYMPOSIUM

Discussion

BY MESSRS. C. S. JARVIS, AND JOSEPH JACOBS

C. S. JARVIS,[59] M. AM. SOC. C. E. (by letter).[59a]—It is indeed gratifying to note the progress achieved recently toward recognizing and sponsoring flood control, stream regulation, and water utilization as both national and local problems. It has long represented "a consummation devoutly to be wished", and earnestly sought by leaders in the Engineering Profession, by far-sighted and progressive economists, legislators, and community officials. Now that such generous provision has been made for the flood control problems in several regions of the United States, it is desirable to continue such activities until equitable distribution has been accomplished among the several States.

It has long been advocated, although occasionally challenged by those reluctant to "fall into line," that all storage development under intelligent operation should result in positive, measurable benefits to flood control and stream regulation. Notable confirmation is accorded in a recent paper by Eric Chester Hillman, with discussions, entitled "The Effect of Flood Relief Works on Flood Levels Below Such Works".[60] Among the conclusions[61] is the following: "The effect of eliminating storage capacity will be to accelerate the flood wave and increase the flood peak below the area from which the floods have been excluded."

In the case of the Nottingham flood protection plan, the reduction of about 10 000 acre-ft of flood-plain storage, due to new protecting levees, resulted in speeding up the flood crest by 7 hr, and a rise of 0.4 ft during

NOTE.—This Symposium was presented at the Fall Meeting of the Society and at the meeting of the Waterways Division, Pittsburgh, Pa., October 13 and 14, 1936, and published in March, 1937, *Proceedings*. Discussion on this Symposium has appeared in *Proceedings*, as follows: June, 1937, by Messrs. F. C. Scobey, Howard T. Critchlow, T. T. Knappen, M. C. Tyler, Gordon R. Williams, Arthur T. Safford, W. G. Hoyt, J. D. Arthur, Jr., John H. Meursinge, H. K. Barrows, E. D. Hendricks, and Edward W. Bush; and September, 1937, by Messrs. H. K. Barrows, Ivan E. Houk, and John E. Field.

[59] Hydr. Engr., Soil Conservation Service, Washington, D. C.

[59a] Received by the Secretary August 9, 1937.

[60] *Journal*, Inst. of Civ. Engrs., Vol. 2, 1935–36, p. 393.

[61] *Loc. cit.*, p. 409.

a freshet, with maximum flow of 20 000 cu ft per sec. Obviously, increase of storage, therefore, would retard and reduce flood crests proportionately.

Conversely, the Bon Accord Dam,[62] impounding 400 000 000 cu ft (9 183 acre-ft) at full supply level, with a surface area of 614 acres and maximum storage depth of 45 ft at the spillway level, reduced a maximum inflow of 16 000 cu ft per sec to a regulated outflow not exceedingly 10 500 cu ft per sec for the maximum observed flood from 121 sq miles, including Pretoria, South Africa.

Even if, in an extreme case, a reservoir may be full or nearly full to spillway level at the arrival of a flood crest, there is generally considerable regulation available due to storage capacity above the spillway crest, together with back-water effects along contributing stream channels. Ordinarily, intelligent operation of storage reservoirs will provide nearly maximum regulation during the critical seasons, and satisfactory flood routing in the lower courses.

JOSEPH JACOBS,[63] M. AM. SOC. C. E. (by letter).[63a]—Every member of the Society who feels a concern in Federal flood-control activities and policies must have read this highly informative Symposium, with great interest and appreciation. Because of the persistent, and not infrequently unnecessary, encroachment of the works of Man upon the flood planes of the rivers of the United States, and because of the increased property values attaching to urban, suburban, and rural developments along its streams, floods are a steadily growing menace and, year by year, are becoming a more serious economic problem both nationally and locally. The writer has always felt a deep interest in the matter and, abstracting largely from material which, from time to time in the past, he has prepared on the subject, he submits the following as an expression of his views concerning a proper approach to the problem of a permanent Federal policy in respect of flood control.

In the discussion of flood control there is frequent reference to the so-called 308 Reports, prepared under direction of the Corps of Engineers, U. S. Army, and it is not an uncommon view, even among engineers, that these reports provide all the preliminary information needed concerning American rivers and that they constitute an adequate basis for the formulation of a national flood-control policy. Commendable as was the undertaking, and excellent as was its execution, it is believed that the 308 Reports were by no means exhaustive finalities as to the matters with which they dealt; nor were they adequately inclusive territorially. This is particularly true as to flood control which apparently received far less consideration than did the navigation, power, and irrigation phases of the reports. No one familiar with the matter, and least of all, the Army engineers, will contend that the reports were in any sense final. Neither sufficient funds nor sufficient time was available to cover the country fully.

[62] *Journal*, Inst. of Civ. Engrs., Vol. 2, 1935–36, p. 432.

[63] Cons. Civ. Engr., Seattle, Wash.

[63a] Received by the Secretary August 30, 1937.

Necessarily, there were many streams that must have been only cursorily investigated and many others, no doubt, that were not then, and have not yet been, investigated at all. It is believed, too, that the 308 investigations were not directed toward the specific problem of a permanent Federal policy as to flood control but dealt rather with the engineering aspects of individual projects. There is yet much important investigational work that ought to be done before any final Federal policy should be formulated in detail.

During the next thirty or forty years probably as much as $2 000 000 000 will be expended on flood-control works and, unquestionably, a large part of this expenditure will devolve upon the Federal Government. Certainly, an activity of that potential magnitude is worthy of the most careful consideration by the responsible agencies, in order that sound technical and legislative policies respecting it may be established and that maximum economy and facility in planning, in execution, and in final operation of the completed works, may be achieved. The evolvement of these governing policies should be based upon definite knowledge of what the flood-control problem is in all its major requirements and ramifications, and when the necessary surveys have been made and the factual data assembled they should be interpreted, in their broad economic aspect, by men technically competent for such a task.

Congress, under the Commerce Clause of the Constitution, assumes jurisdiction over the navigable waters of the United States and generously provides for navigation requirements. To the extent that floods have affected navigation the Government has regarded them as being "affected with a Federal interest" and has made appropriations therefor on that basis. As to floods on non-navigable streams, these, in general, have been regarded as lacking in "Federal interest" and, therefore, have not been considered a matter of Federal concern or responsibility. The Federal Government seems now to recognize that it must retreat from the negative policy that heretofore has characterized its attitude toward flood control and assume a larger responsibility concerning it. This salutary change of attitude is manifest in the Flood Control Act of 1936 and, although the general flood-control measure of 1937 failed of final enactment, the new Federal purpose of a larger responsibility in flood-control matters was also manifest in that measure. The entire subject of flood control is now in a state of flux, and it needs sane guidance toward a policy not temporary or ephemeral, not based on hasty considerations, but one that is sound and enduring because it is based upon a thorough-going investigation of all phases of the problem. No such thorough investigation has yet been made.

The chief significance of the Flood Control Act of 1936 is not so much in its detailed provisions, some of which are deemed open to question, nor in its project authorizations, but rather in the incidence of a new Federal attitude toward flood control. It is the writer's feeling that some of the provisions of the Act are not well considered. It appears to him that the measure must have been rather hastily drawn, probably with

greater consideration for providing early unemployment relief than with a view to formulating a definite, Federal flood-control policy. Those provisions of the 1936 Act, the desirability of which the writer questions, are as follows:

(*a*) It authorizes the appropriation of $310 000 000 for the construction of a very large number of specifically designated projects. It is seriously doubted whether there could have been sufficient investigation to warrant a Congressional approval of the large number of projects involved in this authorization. Presumably, the projects are subject to further investigation and presumably, too, for those that are finally found to lack merit, the actual appropriation will not be made; but is such wholesale approval in advance of final investigation a desirable Congressional procedure? The writer thinks not.

(*b*) It provides that the local agency concerned shall, up to 50% of the total project cost, furnish all required lands and rights of way. The usual defense of this provision is that it will result in lesser right-of-way costs, but that need not be the case if the local agency is required to pay its full and fair share of all project costs, whatever they may be. Rights of way constitute as legitimate a part of a project as do its structural elements, and its cost should not be treated as a thing apart but should be included with other costs without distinction. The inconsistency of the present provision is obvious when one realizes that right-of-way costs may easily range from practically nothing to more than one-half of the total project cost and they cannot, therefore, except by accident, be a fair measure of that portion of the total cost that should be borne by the local agencies. There are many projects for which the local agencies should bear the major part of the total cost and others for which they should bear practically none of the cost, and these facts should be recognized in making cost allocations. An instance is recalled of a $2 500 000 projected Federal institution whose location was largely predicated on a free furnishing of the land required for its accommodation, land that could cost scarcely more than 3% of the total project cost, and the communities of three States were asked to submit offers. How impractical and archaic it is to determine the location of an important Federal enterprise on such a basis! It is to be hoped that the Federal Government will avoid making such a procedure part of its flood-control policy.

(*c*) It provides that the local agencies shall hold the United States free from damages due to the construction works and that they shall maintain and operate all the works after completion. Why should these onerous responsibilities necessarily be imposed upon the local agencies? It seems to the writer that the allocation of these responsibilities and their costs, as well as the allocation of initial construction costs, should be predicated on the theory of conferred benefits. If the Federal interest in a project is dominant, as it might easily be on a navigable stream, or if the project is of an interstate or international character, as some of them will be, then it would seem most practical for the Federal Government to

operate; but this does not necessarily mean that the Federal Government should bear all the costs of operation. Other conditions may clearly indicate that operation should be by the local agency and a final flood-control policy should contemplate and provide for these variable contingencies.

As stated previously, the writer thinks that there is much investigational work yet to be done before a final flood-control policy should be fixed. There are deficiencies in factual information and there are differences in judgment as to the best Federal policy respecting flood control. These deficiencies cannot be supplied, these differences cannot be composed, and the best final policy cannot be determined, without additional facts developed from actual further surveys and investigations. It is suggested, therefore, that the Federal Government ought now to undertake a comprehensive, nation-wide, flood-control survey and investigation, and that the formulation of a final flood-control policy should await the results of that investigation. Some of the factual material that such an investigation should provide, and some of the questions it will have to deal with, are embraced in the following items:

(1) Sufficient basic data to afford a broad-gage perspective of the entire problem including, for all important drainage basins and for the country as a whole, the extent of the flood menace, the average annual losses due to floods, an outline of the character of corrective measures that will need to be applied, and something of an estimate as to the probable gross initial and gross annual cost of these corrective measures.

(2) A careful assembly of all dependable data as to actual flood losses. The validity of flood-control economics is so closely related to a correct evaluation of this factor that a special effort should be made to secure authentic information concerning it. Many wholly unreliable estimates of flood-control losses are extant, both above and below their true value.

(3) An appraisal of the causes of the present unpreparedness for major floods. To what extent is a deficiency of hydrologic data a contributing cause and to what extent is there a failure adequately to heed the warnings afforded by the data already available? To what extent is there unwarranted encroachment, beyond real economic requirements, on the natural flood planes of American rivers? To what extent is there a lack of adequate legislation, both Federal and State, and a lack of an adequate plan of finance, to make possible the construction and operation of needed flood-control works?

(4) What principles should govern in framing a Federal flood-control law? Should such a law provide for, and require, the co-operation of local agencies and provide for an allocation of cost on the basis of benefits conferred and, if so, should this allocation apply only to construction cost, or should it apply also to the initial investigation cost and to the cost of operation and maintenance? Should not the law be devised to serve either a single or a multiple political jurisdiction; that is, should it not

be sufficiently flexible to permit the construction of works for a restricted area or for an entire drainage basin that may extend beyond the boundaries of a single county or a single State, or even beyond a national boundary as might be the case for international streams?

(5) What is the best basis for a rational and equitable appraisal of benefits to Federal and local agencies resulting from the construction of flood-control works? This is not a simple matter but, no doubt, some general principles can be evolved that will facilitate solution for each particular case.

(6) What agency should maintain and operate the flood-control works and what principles should control in determining this question?

(7) Should there be State legislation permitting a requirement, by the State, that certain storage capacities be reserved for flood-control purposes in any new storage reservoirs that may be built, the extra expense entailed thereby to be borne by the beneficiaries concerned?

Doubtless there are other pertinent items that could be added but the foregoing list is sufficiently extensive to indicate that there is a considerable body of factual data and material to be acquired, and that the proposed comprehensive investigation is truly an important and necessary preliminary for the best formulation of a final flood-control policy. It is the writer's judgment that the cost of such an investigation will approximate \$25 000 000 and that a time interval of five years may be required for its completion. It should be emphasized, however, that the prosecution of this work need not in any way embarrass or delay the construction of individual projects that may require earlier attention, but the legislation authorizing them should not be regarded as representing any final Federal flood-control policy.

What provision should be made for prosecuting the proposed investigation? An interpretation of the data provided by the investigation, and the translation of such data into a sound, Federal flood-control policy, will require not only a high order of technical skill and understanding but, equally so, a high order of statesmanship. It is suggested that there be created a Federal Flood Control Commission whose duty it shall be to direct the investigation, and to render a report of its findings, which shall include a specific recommendation as to a definite Federal policy respecting flood control. The personnel of such a Commission should include representatives of those few Federal agencies that now have a major interest in flood control, and it should also include men from the outside who, by training and experience, have special qualifications for such an inquiry. It is the writer's belief that such a comprehensive investigational program as here proposed would, in material degree, receive the financial, co-operative aid of the States.

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DISCUSSIONS

PRESSURES BENEATH A SPREAD FOUNDATION

Discussion

By Messrs. G. P. Tschebotareff, and A. Hrennikoff

G. P. Tschebotareff,[31] M. Am. Soc. C. E. (by letter).[31a]—An interesting review of the most important recent developments in the study of pressures beneath foundations, including a method for the graphical determination of pressures within a soil mass, is contained in this paper. Special attention is given to the so-called "concentration factor." One of the welcome features of the paper is a list of questions, related to pressure distribution studies, which define the problems in need of elucidation by further research work. Most of the branches of soil mechanics and foundation studies have reached a stage of complex development calling for closer co-operation between the various research centers and practicing engineers if further progress is to be made rapidly. Such co-operation might be advanced considerably by a clear formulation of the problems requiring solution in the various fields. The publication of detailed lists of such questions should help to co-ordinate the efforts of the individual investigators and to stimulate them.

The purpose of this discussion is to add a few more points to the outline of the necessary further research work, as given by Professor Krynine, and to offer some comments concerning the possible methods of approach to the solution of these problems.

Problem 1.—When Should the Laws of Pressure Distribution and When —Alternatively—Should the Probable Irregularities in the Compressibility of a Soil Be Adopted as a Basis for Foundation Design?—All estimations of pressure distribution are based on the assumption that the soil is a homogeneous material of uniform compressibility. Systematic investigation is required in different localities to ascertain what types of soil are, as a rule, uniform—at least horizontally—so that the foregoing assumption can be considered sufficiently valid for purposes of practical design. Very little data are available on this important point. In addition to pressure-

Note.—The paper by D. P. Krynine, M. Am. Soc. C. E., was published in April, 1937, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1937, by Messrs. O. K. Fröhlich, Donald W. Taylor, and Jacob Feld.

[31] Asst. Prof. of Civ. Eng., Princeton Univ., Princeton, N. J.

[31a] Received by the Secretary July 28, 1937.

measuring cells embedded in the foundations, this investigation could best be done by systematic settlement observations on numerous points of all new structures to be erected on compressible deposits. The necessity of such observations for all purposes of foundation research has been stressed repeatedly in technical publications by numerous authors, but little has been done actually. It becomes more and more evident that such full-scale observations require the participation of national or local governments, or of other public institutions, to allow their successful organization on a sufficiently large scale.

The writer's experience with settlement observations of fourteen structures in Egypt and some records from other localities to which he has had access, tend to show that there may be in some cases a relation between the geological history of the deposit and the horizontal uniformity of its compressibility. Of course, many more observations and their comparisons are required in different countries before anything definite may be said on the subject, but the following outline may be attempted at present.

Cohesive soils which had been precompressed in the past by previous alternate exposures to drying and flooding showed a fairly regular horizontal distribution of the observed settlements over the area of the structure. This indicated a similarly regular distribution of the pressures within the soil. They appeared to follow a definite law very closely, approaching the one derived from the Boussinesq formulas. Presumably, the previous higher shrinkage and swelling pressures had had a tendency to make these clays and silts more homogeneous in respect of the degree of their resistance to the later smaller pressures from the structures. This may be tentatively explained, as follows: The shrinkage limit, that is, the density down to which a soil is compressed by drying, is fairly low for all fully inorganic cohesive soils and varies over a comparatively small range. (For the Nile clays and silts of the cases referred to the water content at the shrinkage limit varied from about 14% to 28% of the weight of the solid substance.) The soil which has shrunk most will also tend to expand most when wetted again. In the early stages of the life of a deposit of the type considered, adjoining zones of unequal compressibility may exist in the same layer since its sedimentation. (The age of the deposits referred to varies from about 600 to more than 2 000 years.) In each cycle subsequent repeated drying and flooding would cause, first, the considerable shrinkage of all zones of any compressibility and the appearance of numerous vertical shrinkage cracks. Then, during flooding, these shrinkage cracks would be silted in and, simultaneously, the soil around these cracks would start swelling. After a few cycles of this kind the swelling would be partly prevented by the newly added soil which had filled these cracks. The zones most compacted by the shrinkage would have a tendency to swell most and, being prevented from doing so, would as a consequence exert a horizontal pressure on the adjoining less compact zones which would be greater than the counter-pressure caused by the lesser degree of swelling of the latter. The effect of this one-sided excess of

pressure should be to cause a further compaction of the weaker zone and an expansion of the stiffer one until equilibrium would be reached gradually, corresponding to a uniform resistance of both zones to horizontal pressure. This repeated process should have a generally homogenizing influence on fairly large areas of a deposit. The degree of its resistance to further vertical compression should thereby be rendered also more uniform.

The only case of a horizontally irregular degree of compressibility, as evidenced by a measured tilting to one side of a uniformly loaded structure erected on a sedimentary cohesive deposit of the type cited, was recorded during this series of observations on a very recent formation (less than 150 yr of existence). Local hydraulic conditions made it appear very doubtful that it could have been subjected during this short period to the homogenizing process of repeated shrinkage and swelling as tentatively described.

Older and deeper lying but non-prestressed cohesive deposits of different origin in other countries exhibited a varying behavior. Some showed obviously irregular settlements, giving evidence that they were governed by the soil irregularities and not by the pressure distribution. Others exhibited a regularity of settlement distribution in plan which indicated a soil material of almost perfectly isotropic compressibility characteristics. The writer has not come into contact with a sufficient number of settlement records of such cases to allow him to make comparisons for the purpose of establishing the factors governing this difference of behavior. He would only like to stress once more the necessity of gathering numerous complete and detailed records on the subject and of making careful comparative studies of the relevant factors. All the achievements of theoretical investigations and small-scale experiments on isotropic materials will have little use in the final analysis unless one knows when Nature permits the application of the results obtained to engineering practice. Only full-scale observations can give a reliable answer to this question.

It may be expected that both cohesive and non-cohesive soils pre-stressed by glacier loads would exhibit in many cases a horizontally uniform degree of compressibility. Whenever the great pressure of a glacier was applied to a large area of an underlying older deposit, in compressing such area the glacier would probably plane or squeeze out the stiffer zones first, producing thereby a homogenizing effect on the later compressibility of the whole layer. The writer has not come in contact with records of settlement observations on such deposits and, therefore, makes the suggestion as a tentative one.

Glacier loads would probably be the only homogenizing force that could act effectively on a granular deposit if it was originally sedimented in an irregularly loose condition. Drying would be of no effect on such soils. All the records known to the writer concerning settlements on non-prestressed loose granular deposits (that is non-compacted sandy loams, sands,

and gravels of a type where appreciable settlements were measured) appeared to give evidence pointing toward a pronounced horizontal irregularity in the degree of their compressibility.

Further numerous field observations may possibly lead to the deduction of reliable criteria for the guidance of engineers—presumably of a geological nature—indicating the types of soil deposits for which designs may be based on laws of pressure distribution and on what deposits probable horizontal irregularities in the compressibility of a soil should be considered primarily. The practical importance of such criteria is obvious.

Problem 2.—The Depth of the Foundation Beneath the Soil Surface Should Have Considerable Influence on the Value of the "Concentration Factor," the Exact Extent of This Influence Requiring Experimental Investigation.—Several attempts have been made to express mathematically, by means of the so-called "concentration factor," the observed fact that stresses in sand are more concentrated beneath the center of the foundation resting on it than would have been the case if the laws of pressure distribution in an isotropic solid body were valid for sand. The difference in concentration of pressures appears to depend mainly on the nature and the state of the soil material beneath the edges of the foundation; that is, assuming that the foundation rests on the soil surface and that the soil consists of cohesionless sand, a lateral yielding, followed by loss of the supporting power of the soil, is known to occur near the sand surface beneath the edges of the foundation. This phenomenon, in turn, causes a greater concentration of pressures beneath the center of the foundation than is the case on a more cohesive type of soil.

The degree of cohesion of a soil appears to be taken by the investigators of the "concentration factor" as the essential criterion for the choice of its value. For the case of a foundation resting on the soil surface the value of the "concentration factor" is generally assumed to vary from $n = 3$ (clays) to $n = 6$ (sands).

However, the weight of the overlying layers may also be important. The resistance to lateral yielding of the soil at the edges of the foundation depends on its shearing resistance. This resistance, in turn, may be due to the cohesion of the soil, or to its frictional resistance, the latter depending on the weight of the overlying layers. One may assume that at a certain depth of foundation beneath the soil surface this frictional resistance to lateral yielding would become so great that a sand would behave like a solid material with a "concentration factor" of $n = 3$. This may be a probable explanation for the case of the bridge caisson on the Rhine, mentioned by Professor Krynine (see Fig. 12), where pressure cells embedded at its base indicated greater counter-pressures of the sand at the edges than at the center of the caisson, as this should be the case for solid materials where no lateral yielding around the edges is possible. In addition to the weight of the upper layers, pressures transmitted by friction from the external side walls of the caisson may have acted here as an extra surcharge on the area around its base.

Small-scale tests made by Dr. O. Faber, in England, on soil reactions against foundation plates, give interesting indirect evidence on this subject.[32] These tests showed that, for a surcharge of 1.46 tons per sq ft around a circular foundation plate 1 ft in diameter (this surcharge representing the equivalent of about 30 ft of overlying soil), the soil reactions against the edges of the foundation plate were equal to about 60% of the average pressure assumed to be distributed uniformly over the entire test-plate area. For the case of no surcharge, the value of the reactions at the edge reached only 20% of the average pressure.

This increase in the value of the soil reaction against the foundation edge from 20% to 60% of the average pressure, caused by the surcharge around the foundation edge, indicates a substantial increase of resistance against lateral yielding of the soil beneath the edges and, therefore, confirms the views expressed herein concerning the considerable influence on the "concentration factor" of the depth of the foundation below the soil surface. Further experimental investigation of the limit and intermediate values of this influence is desirable. Such investigations should take into account the degree of density of the sand, which is also likely to affect the "concentration factor."

Problem 3.—The "Concentration Factor" Formulas Should Express the Physical Fact That the Influence of the Foundation Boundary Conditions on the Pressure Distribution Decreases with Larger Loaded Areas.—The preceding discussion tends to show that the value of the "concentration factor" depends mainly on the extent of the lateral yielding of the soil around the foundation edges; that is, the "concentration factor" should be governed by the conditions at the foundation boundary. The influence of these boundary conditions on the general pressure distribution obviously decreases with an increase of the size of the loaded area. In respect to soil reactions this circumstance is well illustrated by Fig. 11 of the paper, but it does not seem to have been taken into account in any of the "concentration factor" formulas thus far proposed.

It appears to the writer that the value of the "concentration factor" for a stress component, caused by pressures transmitted to the soil by a foundation element near the center of the foundation area, should be different from the value for a stress component, depending on an element near the edges of the area. In other words, a "concentration factor" changing in value with the distance from the center of the foundation toward its edges, should be used when making the integration of the stress components transmitted from the various foundation elements. The graphical method of integration proposed by Professor Krynine might be adapted to provide the possibility for taking care of this further important variable.

Conclusion (a).—In some cases the study of the geological history of a deposit, combined with full-scale settlement observations of numerous structures, may supply a criterion concerning the probable horizontal uniformity of its compressibility and, therefore, also a criterion as to how far future foundation designs may be reasonably based on pressure distribution laws.

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[32] *The Structural Engineer*, London, March, 1933, pp. 120 and 122, Figs. 3 and 5.

Conclusion (*b*).—The estimation by means of the "concentration factor" of the pressure distribution beneath a foundation is more complex than would appear from the formulas so far proposed for this purpose by several authors. These formulas do not take into account variables of which the "concentration factor" is a function and which should strongly affect its values. These additional variables; that is, the surcharge of the overlying soil layers around the foundation and the size of the loaded area itself, might possibly cause, in many practical cases, a pressure distribution in sand approaching closely that of fully cohesive soils ($n = 3$), for which numerous auxiliary tables, computation graphs, and other accessories have already been developed, greatly simplifying and accelerating pressure computations.

Conclusion (*c*).—Much experimental and theoretical research is still required on this important subject, and Professor Krynine is to be complimented on having furthered its investigation.

A. Hrennikoff,[33] Assoc. M. Am. Soc. C. E. (by letter).[33a]—Another step in the development of soil mechanics is marked by this paper, which is a valuable contribution to engineering knowledge. The writer has no suggestions to make in connection with the clear presentation of the author's interesting method, and he will limit himself to the discussion of the assumptions of the theory of a stress concentration factor, on which Professor Krynine's method is based.

It may seem peculiar that, although the new science of soil mechanics deals with the same material as the classical theory of earth pressure on retaining walls, its development has proceeded along quite different lines, and little use has been made of the findings of the old theory. In this discussion the writer will endeavor to correlate the old and the new theories and to examine in detail some of the principles underlying the latter.

The classical earth-pressure theory involves consideration of a material with mechanical qualities that are characterized by the coefficients of friction, $f = \tan \phi$, and of cohesion, c. According to this theory, if at any point in the earth mass a normal unit stress, s_n, exists on any plane, the shearing unit stress on this plane cannot exceed the value,

$$s_s = f\, s_n + c \qquad (34)$$

although it may be less. This formula, known as Coulomb's formula, represents merely a statement of the fact that the maximum value of the shearing unit stress, at all possible on any plane, at any point, in a granular material, may be considered as the sum of two parts, one of which, $f\, s_n$, is proportional to the normal unit stress on the same plane, and the other, c, is independent of it. Coulomb considered the coefficients, f and c, as constant throughout the entire mass of earth, but for this discussion such a limitation is unnecessary, and the coefficients, c and f, will be thought

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[33a] Received by the Secretary September 1, 1937.

of as quantities constant for different planes at the given point, but varying, if necessary, from point to point. There can scarcely be any quarrel with Equation (34) when so interpreted, since one cannot conceive of any cause that would contribute to the shearing resistance in a manner such that it could not be included either in one or the other of the two parts of the binomial of the formula.

Equation (34) shows the fundamental distinction between a solid body and an accumulation of granular particles constituting the soil. Although in the solid body any value of the shearing stress below the breaking point, imposed by the conditions of continuity, is possible, in the granular material there is a definite limit, and as soon as the limiting value of the shear is reached sliding occurs.

As is shown in standard texts[34] this limitation of the shearing stress results in a limitation of the ratio of the principal stresses. In a cohesionless material (that is, in a material for which c is zero) the least ratio of principal stresses, consistent with Equation (34), is,

$$k = \left(\frac{s_2}{s_1}\right) = \tan^2\left(45° - \frac{\phi}{2}\right) \qquad (35)$$

in which s_1 is the major principal stress, and s_2 is either of the two remaining principal stresses. In a cohesive material the limiting ratio, k, depends on c, and on the value of the major principal stress; it is less than Equation (35), and may be as low as zero.

The writer wishes to emphasize the fact that Equation (35) and its counterpart for the cohesive earth are derived by application of the equations of statics to an infinitesimal wedge of earth, and that they are not influenced by the deformations of the material, elastic or otherwise, and by possible variation of the coefficients, f and c, from point to point.

It follows from the preceding discussion that a granular material may act as a solid body only as long as the ratio of principal stresses at any point, found by application of the theory of elasticity, proves to be larger than the least possible value of k; but as soon as the ratio, $\frac{s_2}{s_1}$, tends to fall below k, the particles slip, the stresses re-adjust themselves, retaining the least ratio, k, between them, and the earth mass ceases to act as one solid body. When that happens, the only equations of the theory of elasticity that hold are those based on statics, while the continuity equations become invalid. This phenomenon of slipping must not be overlooked when the equations of elasticity based on continuity, such as the Boussinesq solution, are applied to earth.

The major principal stress in earth under a foundation assumes a value governed by the load and the two other principal stresses—values governed by k. Thus, if a foundation produces a vertical uniform pressure, p, on the foundation bed (an assumption which is not always true), the major principal stress immediately under the foundation is vertical and equal

[34] For example, "Earth Pressure, Retaining Walls, and Bins," by the late William Cain, M. Am. Soc. C. E., New York, John Wiley & Sons.

to p; and the other two principal stresses acting horizontally are equal to $k\,p$, unless a larger value is required by the conditions of continuity. Minor principal stresses less than $k\,p$ are impossible because that would require the existence, on some planes, of a coefficient of friction greater than $f = \tan \phi$.

With these principles in mind the question of the admissibility of the Boussinesq solution will be considered. This will be done first in relation to the two-dimensional problem, and, subsequently, some remarks will be added concerning the three-dimensional problem.

The solution has been derived for a solid body obeying Hooke's law. According to this solution the major principal stress acts in a radial direction (see Equation (3)) and is equal to,

$$s_1 = \frac{2\,\bar{p}}{\pi\,\rho} \cos \alpha \ldots\ldots\ldots\ldots\ldots\ldots\ldots (36)$$

and the remaining principal stresses, s_2 and s_3, are equal to zero. Whether or not this solution is applicable to earth depends on the following two questions: (1) Does the earth obey Hooke's law; and (2) can the minor principal stresses be equal to zero? Only in a case of positive answers to these two questions can the Boussinesq solution be applied properly to earth.

As to the first of these conditions, the applicability of Hooke's law is conceded by most investigators. However, it must be remembered that only a part of the total deformation may be subjected to this law, namely, the part that occurs before any sliding takes place. The part of the deformation that accompanies sliding, and involves a change of shape without any change of volume, naturally bears no proportional relationship with the stress, since the stress may be considered constant during this phase.

The second condition required by the Boussinesq solution (that is, the vanishing of the minor principal stresses) will now be considered in detail. Three broad cases should be distinguished.

(A).—Cohesionless Material.—In cohesionless material the minor stresses cannot vanish unless the major stress is zero, as is evident from Equation (35); consequently, the Boussinesq solution is not applicable to such material.

(B).—Dry Cohesive Material.—It is possible for the minor stresses in dry cohesive earth to vanish, provided c is sufficiently high and the major stress is sufficiently low. The greatest value of the major stress for which the minor stresses may vanish, can be found from the relation[34],

$$s_1 = 2\,c \tan\left(45° + \frac{\phi}{2}\right) \ldots\ldots\ldots\ldots\ldots\ldots (37)$$

Equation (37) is derived from statics without any special assumptions. A conclusion, then, may be drawn that once the major stress under the action of a line loading exceeds the limiting value (see Equation (37)) the solidity of the earth is broken, which, in turn, means that the Boussinesq solution

does not hold. Taking an example of stiff clay with $\phi = 7°$ and $c = 1\,500$ lb per sq ft,[34] the value of s_1 from Equation (37) equals 3 390 lb per sq ft, a value that is not very high. The writer does not insist on the accuracy of the assumed coefficients for clay in his numerical example, but simply wishes to emphasize the fact that in conditions encountered in practice the dry cohesive earth is not likely to behave as one solid body, at least for some depth below the foundation; but, at a greater depth, where the major principal stress decreases sufficiently, it may act as a solid body. This conclusion is corroborated by the testimony of several investigators, referring to the region immediately beneath the foundation as a disturbed zone. From the writer's point of view this zone is merely a part of the earth mass in which the particles have undergone some sliding, and in which, consequently, the theory of elasticity does not hold.

(*C*).—*Cohesive Earth Saturated with Capillary Water.*—If the foundation bed of a cohesive earth is situated between the levels of capillary and gravitational waters, a state of uniform pressure due to a capillary action is superimposed on the state of stress caused by the load, increasing by the same amount both the major and the minor principal stresses. This condition is favorable to a behavior of earth as a solid continuum, and obviates the necessity of forming a disturbed zone. However, in order that the capillary pressure may properly materialize, the earth must be nearly saturated with capillary water. The writer does not think that mere presence of moisture in a cohesive earth would make it behave in a manner essentially different from dry earth, although the coefficients, c and f, may be affected by moisture content. No capillary pressure is present below the level of gravitational water. Capillary action in sand and gravel is negligible.

Failure of the Boussinesq solution to agree with the experiments led the investigators to adjust it by an arbitrary introduction of a quantity, $(n - 2)$, instead of unity in the power of the cosine in the expression (Equation (36)) for the radial stress. The coefficient, 2, also had to be changed into $n_1 = f(n)$ due to considerations of statics; thus, the author's Equation (6) was obtained. In this empirically modified Boussinesq solution all other results of the original theory are retained; the radial stress is still considered as the major principal stress, and the two other principal stresses are held equal to zero; the empirical constant, n, termed the concentration factor, varies in magnitude depending on the nature of soil material. It is this modified theory that has been used by the author in the development of his graphical solution.

The weakness of the modified theory lies in its failure to take proper account of the earth disturbance, a fact leading to several apparent inconsistencies:

1.—As has been demonstrated, the minor principal stresses, caused by the loading, may be equal to zero only in cohesive material when it is lightly loaded, or nearly saturated with capillary water. When such a material is nearly dry, or completely submerged below the gravitational

water level and, at the same time, is fairly heavily loaded, the minor stresses do not vanish, and it does not seem justifiable to disregard them. In cohesionless material the minor stresses cannot be equal to zero under any circumstances. One might think that this criticism would not apply if several line loadings were superimposed; however, the objection would still hold, at least for the region near the boundary of the loaded area.

2.—It seems improbable that the same value of n should properly describe the state of stress both near the foundation where the soil is disturbed and outside the zone of disturbance. The writer feels that the stresses at a point depend not only on the geometric location, but also on whether the point in question lies inside or outside the disturbed zone. It also appears that the size and shape of such a zone should have an important bearing on the settlement of the foundation and on the law of distribution of pressure over the base.

3.—The use of the principle of superposition in conjunction with the modified Boussinesq theory seems incorrect. As has been stated, the introduction of the empirical exponent, $(n - 2)$, was due to one of two causes: (*a*) The failure of the earth to obey Hooke's law; or (*b*) earth disturbance. Either one precludes the application of the principle of superposition. That the statement is true with respect to the first reason needs no explanation. The earth disturbance, however, has the same effect: First, because the boundary of the disturbed zone is not the same for the three loads—Loading 1, Loading 2, and Loading $1 + 2$; and second, because in the disturbed zone, the minor principal stresses do not combine vectorially, but keep a constant ratio, k, with the combined major stress.

It may be argued that superposition is an additional empirical assumption which is to be used with the empirical formulas of the modified Boussinesq theory, but such an assumption stands as a contradiction to the physical nature of the phenomenon which it purports to describe. Suppose, for a moment, that Equation (6), describes truly the state of stress in certain soil under a line loading, and that the value of the concentration factor is determined from experiments with such a loading. If then the same soil is subjected to several line loadings, and the state of stress is analyzed by the formulas derived by superposition which involve the same value of n, then a disagreement will be noticed between the observed and the computed values of stress. The disagreement may be removed by a proper modification of the value of n accompanied by an admission that the magnitude of the concentration factor depends not only on the character of the soil, but also on the character of the load and on the depth. Thus, the assumptions of the modified Boussinesq theory lead to a deduction of variability of the concentration factor, with its dependence on several variables besides the character of the soil. This complicates the matter considerably. The difficulty, of course, is the natural consequence of an attempt to describe a physical phenomenon by an uncongenial formula and an unsuitable assumption.

The writer feels that the criticism of the use of superposition in the modified theory strikes at its very root, and unless it is removed, the theory fails, since superposition is the very essence of its application.

4.—The reasons for the particular significance attached by most investigators to p_z, the normal unit stress on the horizontal plane, are not clear. The primary object of stress analysis in the earth under a foundation is the determination of the foundation settlement, caused by the earth deformation, but this deformation is manifestly a function of all three principal stresses, and not of p_z alone. Nor can p_z always be considered nearly equal to the major principal stress. The writer thinks that the pressure exerted by the actual foundation on a bed of cohesionless material may be inclined at an angle of friction with the normal. If that is true, the major principal stress over a large part of the foundation bed in sand and gravel must be far from vertical.

The foregoing criticisms have been made primarily as they apply to the two-dimensional problem. They also apply with minor qualifications to the three-dimensional problem, in regard to which the writer wishes to make the following additional comment:

5.—In the original Boussinesq solution the radial stress is not a principal stress, except for an incompressible elastic solid body, and none of the principal stresses is zero. In the empirically modified solution, Equations (1) and (4), the radial stress is treated as the major principal stress and the other principal stresses are considered as non-existent. Although the author does not make any use of this interpretation, if it is adopted the original Boussinesq solution ceases to be a special case of the modified theory, except for the aforementioned physically impossible material. From these considerations it follows that if a certain earth material exhibits, in its mechanical characteristics, a great similarity to an elastic solid body, such a material is not subject to analysis by the modified Boussinesq theory, a conclusion which is as unexpected as it is logical. The modified theory, therefore, must be supplemented by the list of soils to which it is applicable.

Conclusion.—The Boussinesq theory has been chosen as a starting point in soil mechanics by many investigators. After some adjustments, an apparent agreement with the experiments has been reached, and the investigators have been eager to proceed with further development, whereas the nature of underlying assumptions apparently has received only casual attention. This discussion, by emphasizing some seeming inconsistencies, is intended to bring forward the necessity for critical examination of the basic principles on which the theory rests. The writer appreciates the qualitative value of the original Boussinesq theory in describing the stress behavior of the earth mass, and is inclined to concede a limited field for its applicability in quantitative determination of stress when dealing with cohesive materials, but he feels unable to explain the numerous apparent contradictions inherent in the modified theory. If the writer's diagnosis is correct, and the contradictions indicated by him are irreconcilable, a further elaboration of the theory of the concentration factor is futile.

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DISCUSSIONS

EFFECT OF DOWEL-BAR MISALIGNMENT ACROSS CONCRETE PAVEMENT JOINTS

Discussion

BY MESSRS. L. W. TELLER, DAVID J. PEERY, AND L. J. MENSCH

L. W. TELLER,[3] ASSOC. M. AM. SOC. C. E. (by letter).[3a]—A subject that is of importance to every designer and builder of concrete pavements is treated in this commendable paper. The data which the authors furnish from measurements of dowel positions in place, are useful in demonstrating, rather conclusively, what is more or less common belief—that, frequently, good dowel-bar alignment is not obtained in actual construction. The effect of such misalignment upon dowel efficiency is a matter that should receive very careful consideration before an attempt is made to draw broad conclusions or to recommend general tolerances for field construction. Unfortunately, the authors did not include all the data from the experiments that were used to determine the permissible errors that are recommended, and, therefore, no detailed discussion of their test data is possible. Some general discussion of the subject of dowel efficiency is pertinent, however.

A dowel-bar installation really has two functions in a concrete pavement: First, it prevents any gross faulting of the two slab ends and preserves the general continuity of the pavement surface at the transverse joint; and, second, through its shear resistance, it assists the slab end in carrying wheel loads by transferring a part of the load across the joint to the adjacent slab. The installation requirements for the dowels, in order that they may perform these two functions, are of two entirely different orders of precision because the order of the vertical slab movements with which each is concerned is essentially different. A dowel installation that will perform the first function satisfactorily may not perform the second function at all. It is important to realize why this is so.

NOTE.—The paper by Arthur R. Smith, M. Am. Soc. C. E., and Sanford W. Benham, Assoc. M. Am. Soc. C. E., was published in June, 1937, *Proceedings*. This discussion is printed in *Proceedings*, in order that the views expressed may be brought before all members for further discussion of the paper.

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[3a] Received by the Secretary July 6, 1937.

When a normal heavy wheel load approaches the end of a concrete pavement slab, the slab deflects, but for slabs of the usual design the deflection is quite small. Tests[4] have shown that these deflections may be expected to be of the order of 0.020 in. to 0.030 in. for a free slab end. This deflection may be reduced, through the structural action of the joint, by possibly 0.010 or 0.015 in. Thus, it is apparent that whatever the single dowel is to accomplish must be done during a relative displacement of the two slab ends of only a few thousandths of an inch. The bearing of the dowel on the concrete must be maintained within close limits if the unit is to develop shear resistance and thus transfer load.

In their "Synopsis," the authors state quite correctly that it is important to place the bars accurately. If they are not perpendicular to the cross-section of the pavement, binding between them and the concrete will occur when the joint opens or closes. As a consequence, "either the bars will become distorted or the concrete will be damaged in the vicinity of the bars."

The bearing stress imposed on the concrete by the round steel dowel-bar is very high, and it is the concrete that fails first. The type of failure known as "funneling," observed under certain conditions in the concrete immediately around a dowel, is the direct result of repeated applications of this high bearing stress.

Since the effectiveness of a dowel can be lost entirely if a relatively small amount of "play" is developed between it and its socket, it is apparent that, in the tests reported by the authors, the absence of structural cracking or visible spalling does not necessarily mean that the dowels retained their effectiveness so far as their function of transferring load is concerned.

When a wheel load rests on a slab end at a distance from a corner the critical stress in the concrete is a tensile stress in the bottom of the slab directly under the wheel load. The direction of this critical deformation is parallel to the edge of the slab. The load-transfer feature of a joint installation is intended to relieve this stress and the degree to which it accomplishes this relief is a measure of the efficiency of the joint action.[5] The only apparent way in which one can determine joint efficiency (or change in joint efficiency) is by means of stress determinations that will show, directly, the degree to which the joint is fulfilling its function of relieving the critical edge stress. The authors do not report tests of this kind and without them the effect of dowel alignment on dowel efficiency remains unknown.

The authors do not present the data on the forces required to produce slab displacements for various degrees of misalignment, but state that in no case did they exceed 4 000 lb per bar. From this they reason that the loads "seemed to be of no practical significance." As far as compressive stress is concerned this may be true, but if poor dowel alignment can cause

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[4] "Structural Design of Concrete Pavements"; Pt. 4, "A Study of the Structural Action of Several Types of Transverse and Longitudinal Joint Designs," *Public Roads*, Vol. 17, No. 7, September, 1936, p. 166.

[5] *Public Roads*, Vol. 17, No. 7, September, 1936, p. 152.

tensile stresses of the order of 50 lb per sq in. in the concrete, then the stress has practical significance. In certain parts of the slab, loads and temperature warping produce tensile stresses at a common point. If joint resistance adds to this, it subtracts directly from the stress resistance available to carry wheel loads. A properly installed plain round dowel should not develop more than about 100 lb of total resistance to longitudinal movement, which is equivalent to about 1 lb per sq in. of concrete stress.

It is not the writer's purpose to criticize the authors nor the tests that they made, but rather the generality of the conclusions which they draw. Until such time as tests are made that measure directly the effect of misalignment of dowels on the structural efficiency of the joints in which they are installed designers should bend every effort toward the development of means for installing them as perfectly as possible and not encourage careless installation in the field by recommendations such as are made in this paper.

DAVID J. PEERY,[6] JUN. AM. SOC. C. E. (by letter).[6a]—An investigation of the alignment of pavement dowels supplies information for studying the efficiency of the dowel bars in transferring loads across joints. A homogeneous slab of uniform thickness, supported on an elastic subgrade, will be much more highly stressed by a load applied at a free edge of the slab than by the same load applied at the interior of the slab. A balanced slab design, in which the maximum stresses at the edges are equal to the maximum stresses in the interior, is approached by thickening the edges or by providing dowel bars to transfer part of the load to the edge of adjacent slabs.

If dowels are strong and stiff enough to cause equal deflections of the two slabs at the points of connection, they will transmit about one-half the total load applied at the edge of one slab. This condition, however, does not reduce the critical stress in the loaded slab by one-half, but it does result in a considerable reduction if the dowels are spaced closely. With information available as to the vertical misalignment of dowels in pavements representing common construction practice, it is possible to investigate the effectiveness of dowels in deflecting the points of connection equally.

For effective dowel action the difference in deflections between the edges of the two slabs at the points of connection must be very small in relation to the deflection of the free edge of one slab under a wheel load. The maximum deflection of a free edge of a homogeneous slab under a concentrated load at the edge may be determined by an equation,[7] proposed by H. M. Westergaard, M. Am. Soc. C. E.:

$$z_0 = 0.434 \frac{P}{k\, l^2} \qquad (1)$$

in which, z_0 = maximum slab deflection; P = wheel load; k = subgrade modulus; and, l = radius of relative stiffness. For a wheel load of 10 000 lb

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[6a] Received by the Secretary July 14, 1937.

[7] "Spacing of Dowels," Highway Research Board, 8th *Proceedings*, p. 156.

at the edge of a 7-in. slab, with k = 200 lb in.$^{-3}$, and l = 25.73 in., the maximum deflection is 0.0327 in. This value will be used only to represent the general magnitude of the deflections. The relative deflections between any two dowels would be much smaller.

An investigation of a series of adjacent dowels with similar vertical misalignments will show what relative vertical displacements occur when a joint opens or closes. Assuming a vertical error of 1 in. in installing 22-in. dowels, and an expansion joint movement of 0.75 in., a relative vertical displacement will occur equal to $\frac{1}{22} \times 0.75 = 0.0341$ in. Since this is even larger than the deflection caused by the load, the dowels do not aid in reducing stresses by load transfer. The nature of this distortion is shown in Fig. 10. If a load is placed on either side of the joint, the deflection of the slabs tends to return the left slab to its unstressed position, and most of the load is carried by the slab on the right.

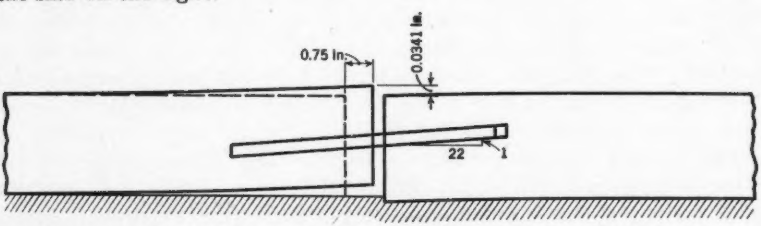

FIG. 10

It is evident that dowels which are this much out of line are objectionable because they fail to transfer load. They also cause stresses in the slab by a wedging action, and they resist movement at the joint. If adjacent dowels are inclined in opposite directions, any movement of the joint will cause local over-stresses and elastic failures. Although these may or may not cause complete failure after several cycles of reversal, they will produce enough yielding and play in joints so that effective load transfer is lost.

Any type of pavement joint which transfers load should be designed and installed with great care. As slab deflections under loads are usually less than $\frac{1}{32}$ in., any load-transfer device must be constructed so as to permit very little relative vertical movement between the connected edges of the slab.

L. J. MENSCH,[8] M. AM. SOC. C. E. (by letter).[8a]—The art of making joints in concrete construction in general and in pavements in particular is still in the stage of discovery and invention, and is a lucrative field for the patent lawyer and the mountebank. Although many millions of dollars are expended annually on joints, the development of the art has covered a long period of time, and "the trade" has been subject to the losses, faults,

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[8a] Received by the Secretary August 4, 1937.

and checks to which haphazard knowledge, without adequate tests and proper analysis based on them, is peculiarly liable.

The specifications of State highway departments for joints have lagged seriously behind the best accepted rules of structural engineering and concrete practice and have acted as a deterrent to more scientific development by independent manufacturers of joints. The successful designer of joints must have an unusually thorough training, and long and wide experience, if he is to comprehend and provide successfully for all the variation of stress in dowels and concrete due to many influences, such as warping of the slabs from wet sub-grade or from temperature variation, misalignment of dowels, and movements of the slabs as a whole, vibrations of slabs, cavities at the joints caused by the pumping action of traffic on the water-soaked sub-grade, heavy wheel loads on one side of the joint, etc.

The most satisfactory pavement would be one without joints; this, however, does not seem possible of attainment as the aforementioned influences cause considerable irregular cracking in every direction. The Bates Road tests in Illinois, and the Pittsburg, Calif., tests of fifteen years ago have taught American engineers that longitudinal joints greatly prevent irregular longitudinal cracks.[9] Nothing so clear was learned by these tests, about transverse joints. In the years following, most pavements were constructed with tongue-and-groove longitudinal joints, thickened edges, and transverse joints rather far apart. After five years these pavements showed many irregular transverse cracks, and after ten years, blow-ups occurred as often as one every three miles, once each year. Early in 1934, the U. S. Bureau of Public Roads required, on all Federal Aid roads, the use of expansion joints not more than 100 ft apart to guard against blow-ups, and the use of contraction joints for crack control. The Bureau also required effective means of load transfer at both expansion and contraction joints, the minimum allowable being the use of $\frac{3}{4}$-in. dowels, 24 in. long, 12 to 15 in. on centers. Even at that time doubt was expressed as to whether $\frac{3}{4}$-in. dowels were sufficiently strong for the purpose.

The Highway Department of Indiana 'like those of many other States which did not specify transverse joints in its pavements, was suddenly compelled to use them, and evidently the authors were astounded when they saw the recklessness and disregard of engineering principles with which dowels were installed in the slabs. They made the tests described in this paper and did a very meritorious job. They not only proved: (1) That dowels were not installed parallel to the longitudinal axis of the road when unusual care was exercised and when special installation devices were used; (2) that the dowels were not parallel when they were welded to two $\frac{5}{8}$-in. cross-bars, showing errors as great as 1 in. on account of the bending of the cross-bars in shipping and handling; but (3) they also proved (what is even to-day scarcely realized by many highway engineers) that the concrete operations and the action of the mechanical strike-off machine (mis-named finishing machine) cause a still greater error in the location

[9] "Highway Research in Illinois," by Clifford Older, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 1180; and Report of Highway Research at Pittsburg, Calif., 1921–22, Dept. of Public Works, California.

of the dowels. Any engineer who watches the modern method of placing concrete in pavements must be surprised that contractors are permitted to dump an entire yard of concrete crudely by the nearly uncontrollable opening of the traveling bucket of the mixer, often producing a wave-like motion of the heavy mass which throws joints and anchoring devices many inches out of line. Furthermore, the top of the dumped concrete is often many inches above the top of the pavement and offers a great resistance to the advancing strike-off machine, which again throws the joint out of line, rarely less than 0.5 in. and often as much as 1.5 in. on top. From his many observations of paving operations the writer is convinced that the errors produced by concreting and by the strike-off machine are much larger than those found by the authors.

Some highly speculative studies on the load transfer of dowels have been published. These studies were not based on any tests and evidently did not agree with the findings of the authors; therefore, they assumed that the subject is impossible of analytical solution. Although the task is certainly difficult and unusual, the writer has shown[10] that a rational description of the action of dowels is possible without the use of too much higher calculus and will now offer an improved and simplified analysis.

In Fig. 11 let the verticals through Points A and F represent the original slab faces of an expansion joint with an opening of 1 in., and the verticals through Points A' and B', the faces of the slabs after the joint has been compressed; let Line $A\,B$ be the direction of the mis-aligned dowel, and the distance, $B\,F$, be the error of $\frac{1}{24}$ in. corresponding to an error of 1 in. in 24 in. By closing the joint space, Point A moves to A' and Point B to B' and the dowels would be bent into a broken line, $E\,A'\,B'\,D$, if the resistance of the concrete proved to be greater than that of the steel of the dowel. Since concrete is the weaker material, the dowel will deform according to a smooth curve, somewhat as shown by $A''\,C\,B''$, having a point of inflexion at Point C. This bending can be imagined to be caused by a shear, P, acting at Point C which will produce high compression in the concrete at the under side of the dowel at Point B''. The action of the dowel is very much like that of nails in timber construction. On any reinforced concrete job one can easily observe the shape into which the nails are deformed after they are highly strained, when the lumber is cleaned and assembled for re-use.

The greatest curvature of the nail or dowel is found only a short distance from the Point B', and between the latter point and the point of greatest curvature a permanent deformation of the timber is produced. Point D is about twice as far from Point B' as the point of greatest curvature of the nail or dowel; beyond Point D and up to Point G the nail is nearly straight and does not produce a permanent deformation of the timber. Fig. 11(*b*) shows the probable deformation.

The stresses in dowels and in the surrounding concrete in the part, $B'\,D$, are always high in the tests offered by the authors, being mostly beyond the yield point, and the distribution of the stresses exerted by the

[10] "Joints for Concrete Pavements," Chicago, 1935.

dowel on the concrete in the part, $B'D$, can be assumed to be according to a cubical parabola, as shown in Fig. 11(*c*). The writer has previously shown[10] that the part, DG, behaves nearly like a cantilever, projecting beyond Point D at the angle, α, and bent down by an ideal uniform load

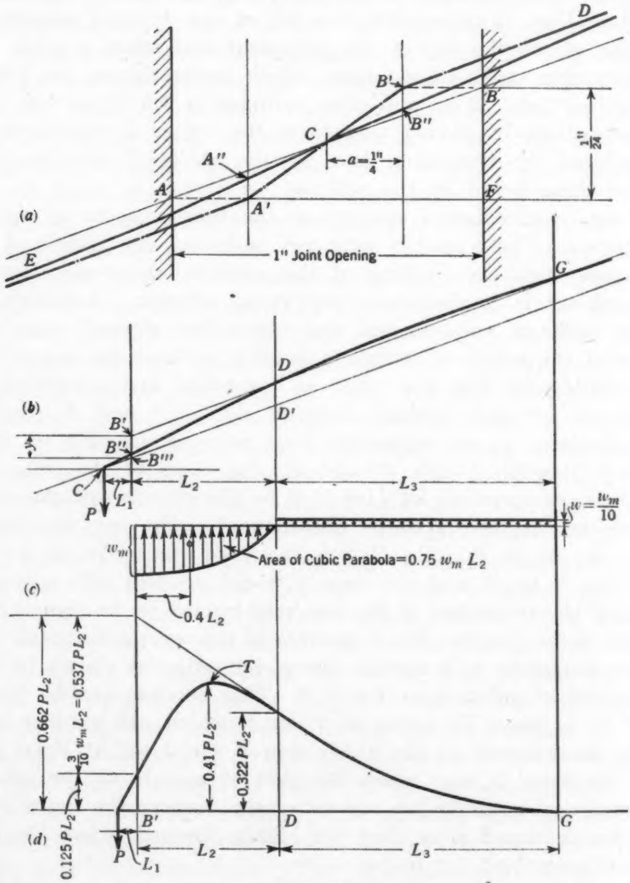

Fig. 11.

w, so that the end of the cantilever reaches the level of Point G. By computing many cases, he found that a good assumption is,

$$w = \frac{w_m}{10} \quad \text{.......................(2)}$$

when w_m is the maximum pressure at Point B'. The dowel can now be considered as a simple beam affected at Point C by the unknown shear, P, loaded in the part, $B'D$, by the upward concrete stresses which are decreased from a maximum value, w_m, at Point B', according to a cubical

parabola, to zero at Point D, and loaded by the ideal uniform downward stresses, $\frac{w_m}{10}$ in the part, $D\,G$; and the following equations can be written:

The sum of all vertical forces must be zero, or,

$$P + w\,L_3 = \tfrac{3}{4}\,w_m\,L_2 \qquad (3)$$

The moments about Point B' must be zero:

$$P\,L_1 + \tfrac{3}{4}\,w_m\,L_2 \times 0.4\,L_2 = w\,L_3\,(0.5\,L_3 + L_2) \qquad (4)$$

From Equation (3) and Equation (4),

$$w_m = \frac{P}{0.75\,L_2 - 0.1\,L_3} \qquad (5)$$

The value of w_m introduced in Equation (4), gives a quadratic equation in L_3 from which:

$$L_3 = -\,(L_1 + L_2) + \sqrt{(L_1 + L_2) + 10\,L_2\,(1.5\,L_1 + 0.6\,L_2)} \qquad (6)$$

The authors advise assuming $L_1 = 0.25$ in.; by assigning to L_2 various probable values, such as 1.5 in., 2 in., 2.5 in., etc., Equation (6) leads to a very close approximation:

$$L_3 = 1.9\,L_2 \qquad (7)$$

and Equation (5) reduces to:

$$w_m = \frac{P}{0.75\,L_2 - \frac{1.9\,L_2}{10}} = \frac{P}{0.56\,L_2} \qquad (8)$$

It is clear that the deflection of the dowel under the assumed loading must be equal to $B'\,B'''$, which, in the present case, is one-fourth of $\frac{1}{24}$ in., or $\frac{1}{96}$ in. An expression for this deflection, y, can be obtained as follows: The angle, α, for the cantilever, $D\,G$, equals[10]:

$$\alpha = \frac{w\,L_3^3}{8\,E\,I} \qquad (9)$$

Introducing into Equation (9) the values from Equations (2), (7), and (8),

$$\alpha = \frac{0.153\,P\,L_2^2}{E\,I} \qquad (10)$$

and the deflection of the dowel at Point C from the rotation, α, is:

$$y_C = (L_1 + L_2)\,\alpha = \frac{0.153\,(L_1 + L_2)\,P\,(L_2)^2}{E\,I} \qquad (11)$$

The deflection of the dowel at Point C from the moments acting in the part, $B'\,D$, is best found by the moment-area method; thus, the moment at Point D is,

$$M_D = \frac{w\,L_3^2}{2} = \frac{P}{5.6\,L_2} \times 1.9^2 \times \frac{L_2^2}{2} = 0.322\,P\,L_2 \qquad (12)$$

and the moment at Point B' is

$$M_B = w\,L_3\,(0.5\,L_3 + L_2) = 0.662\,P\,L_2 \qquad (13)$$

This latter moment, M_B, is diminished by the moment of the upward concrete stresses acting in $B' D$, or,

$$M_{B'} = 0.75 \, w_m \, L_2 \times 0.4 \, L_2 = 0.537 \, P \, L_2 \ldots\ldots\ldots\ldots (14)$$

The deflection of the dowel at Point C from the moment area, $B' D T$, in Fig. 11(d) is the moment area times the distance of its center of gravity from C_1:

$$y_C' = 0.173 \frac{P L_2^3}{E I} + 0.317 \frac{P L_1 L_2^2}{E I} \ldots\ldots\ldots\ldots (15)$$

and the total deflection is found by adding Equation (11) and Equation (15), or,

$$y = 0.326 \frac{P L_2^3}{E I} + 0.47 \frac{P L_1 L_2^2}{E I} \ldots\ldots\ldots\ldots (16)$$

Instead of Equation (16) it will be more convenient to use the approximate formula:

$$y = 0.38 \frac{P L_2^3}{E I} \ldots\ldots\ldots\ldots (17a)$$

and,

$$P = \frac{E I y}{0.38 \, L_2^3} \ldots\ldots\ldots\ldots (17b)$$

in which E equals 30 000 000 and I is the moment of inertia of the dowel (0.0155 for $\frac{3}{4}$-in. dowel). By assuming various probable values of L_2 (say, from 1 in. to $2\frac{1}{2}$ in.) and of y, the deflection produced by the movement of the joint faces ($\frac{1}{96}$ in. in the present case), the corresponding values of the ideal shear, P, can be computed from Equation (17b) and the corresponding maximum pressures, w_m, in pounds per linear inch, which the dowel exerts on the concrete, can be computed by Equation (8). From Fig. 11(d) it may be further noted that the greatest moment exerted on the dowel by the concrete pressures is $0.41 \, P \, L_2$ which values divided by the section modulus of a $\frac{3}{4}$-in. bar (0.0414) give the unit steel stresses in the dowels. The values thus computed are given in Table 2.

TABLE 2.—STRESSES IN DOWELS AND CONCRETE

Description	Values of Length, L_2, in Inches				
	1	$1\frac{1}{2}$	$1\frac{3}{4}$	2	$2\frac{1}{2}$
Shear, P, in pounds, from Equation (17a)	12 600	3 740	2 340	1 560	810
Maximum unit pressure, w_m, in pounds per linear inch, from Equation (8)	22 500	4 350	2 400	1 400	580
Values of $\frac{w_m}{0.75}$, in pounds per square inch, for $\frac{3}{4}$-in. dowels	30 000	5 800	3 200	1 870	773
Moment, $M = 0.41 \, P \, L_2$, in inch-pounds	5 160	2 300	1 680	1 280	835
Steel stresses, S_s, in $\frac{3}{4}$-in. dowels, in pounds per square inch	124 000	55 800	41 000	31 000	20 200

Inasmuch as neither P nor L_2 is given, and as it is extremely difficult to measure either the concrete or the steel stresses, it is very difficult to determine which group of values refer to a particular test. The authors

state that they observed crushing and splitting in the concrete in the case of a 5-in. slab. They probably also found pulverization of the concrete at Point B' in the 6-in. slabs; hence one may conclude that the stresses in a 6-in. slab were also very high.

In Table 2, the column for $L_2 = 1\frac{3}{4}$ in. shows concrete and steel stresses of 3 200 and 41 000 lb per sq in., respectively. These are by no means unlikely values as it is quite common to observe steps in joints due to the movements of the slabs. Such steps indicate very high stresses in the dowels and concrete. When $d = 2$ in., concrete stresses equal 1 879 lb per sq in., and steel stresses, 31 000 lb per sq in., which are still very high values. The results of this analysis will scarcely be changed when one slab is fixed and the other slab moves the entire 0.5 in.

Assuming that more scientific and more extensive tests may demonstrate that correction factors must be applied to all the foregoing equations, they are unlikely to change values more than 30 per cent. This analysis shows that slab movements of 0.5 in. may cause very high stresses in concrete and anchoring devices, as usually installed, and there is very little strength left for the primary function of the dowels and anchoring devices —the transfer of loads. Searcy B. Slack, M. Am. Soc. C. E., made tests[11] which demonstrated that $\frac{3}{4}$-in. dowels did not transfer any load in actual pavements after they had been in service for a few years.

Imitating modern connectors for timber construction several joint manufacturers are using sleeves over $\frac{3}{4}$-in. dowels in order to diminish the high concrete stresses at Point B'. Table 2 enables one to judge their merits. For $L_2 = 1$ in. and a sleeve diameter of 1.5 in., the concrete stresses would be $\frac{22\,500}{1.5} = 15\,000$ lb per sq in., and the steel stresses, $0.41\,P \times \frac{1}{0.0414} = 71\,000$ lb per sq in.; whereas, for $L_2 = 1.5$ in., the stresses would be 2 900 and 47 800 lb per sq in., on concrete and steel, respectively. The sleeves and dowels being very short one is not entitled to assume larger values of L_2. The writer has heard that the authors made tests to study the sliding of such devices and found resistances to sliding four times as great as for $\frac{3}{4}$-in. dowels without sleeves.

In the writer's own tests on sliding of mis-aligned $\frac{3}{4}$-in. dowel bars[10], the resistance to sliding was much greater than that found by the authors on account of the greater stiffness of the high-carbon dowels used. The authors proved by their tests that the slip-shod methods used at present to place and hold dowels and anchoring devices before and after concreting must be abandoned. Positive means are needed to hold dowels, anchoring devices, and the entire joint assembly in correct alignment until the concrete has set.

The closer the expansion joints are spaced, the less is the movement and the less are the stresses caused by mis-aligned dowels and anchoring devices; and it is very probable that in the near future expansion joints in pavements will be placed about 30 ft apart instead of 80 ft to 100 ft as at present, and some kind of inexpensive contraction joint will be used between the expansion joints.

[11] *Engineering News-Record,* July 9, 1931.

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DISCUSSIONS

SOIL REACTIONS IN RELATION TO FOUNDATIONS ON PILES

Discussion

BY MESSRS. GEORGE A. MCKAY, AND CHANDLER DAVIS

GEORGE A. MCKAY,[6] M. AM. SOC. C. E. (by letter).[6a]—An important phase of foundation engineering which has received insufficient consideration, is treated in the paper by Mr. Miller. In contrast with the more exact determination of strength of the various parts of the more familiar superstructures, the fixing of the size and character of foundations has too often been a matter of rough approximation based on the limited information available as to the action of somewhat comparable constructions in somewhat comparable soils. If the structure stood, it was evidence of sound judgment, and if it settled, there was generally a self-satisfying excuse as to unknown, unusual, and unforeseeable subsurface conditions which, when investigated, were given but little publicity.

Although improvement in practice has been steady, there is, and there always will be, a long way to go before engineers can approach the confidence with which other better known and more easily studied elements of structural engineering are treated. In dealing not with pile strengths or pile formulas, but with soil behavior under pile loads, this paper marks a step toward focusing attention on this vital but comparatively neglected factor in safe construction.

There was a time, and not so many years ago, when the average engineer, in the course of designing a medium heavy structure on somewhat questionable soil, would cease to worry over possible serious settlement when he decided to use pile footings rather than spread footings. Such a decision, he felt, eliminated the uncertainties as to soil settlements from spread footings under load and rendered the design exact enough to fall safely within the factor of safety used in standard pile formulas, which with easy calculation convert a penetration from a dynamic blow into a static

NOTE.—The paper by R. M. Miller, M. Am. Soc. C. E., was published in June, 1937, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1937, by Messrs. Hibbert M. Hill, and George P. Stowitts.

[6] Capt. (CEC), U. S. Navy; Public Works Officer, 11th Naval Dist., San Diego, Calif.

[6a] Received by the Secretary August 4, 1937.

safe bearing value. Following such a decision, the remaining foundation design was easy. The estimated bearing value per pile, divided into the total load, gave the number of piles, and the size, shape, and depth of the foundation followed after consideration of other allied economic and easily analyzed conditions.

In general, two vital considerations were overlooked, and both fall within the scope of the paper and are illustrated by examples given therein. One was the limit which could be placed on the supporting power of the soil itself to carry the loads passing either from end bearing of the pile points or from skin friction to the soil surrounding the pile. A false feeling as to the security of pile foundations had gradually developed from observed and published examples applicable to isolated pile behaviors, such as heavy loads supported by pier piles driven on, say, 8-ft to 15-ft spacing in soft mud where such piles safely carried much heavier loads than the standard pile formula indicated would be safe. Engineers knew that piles in the process of being driven were often, in some manner, lubricated, and that, in general, after rest the skin friction increased, and the piles showed higher resistance and increased bearing strength when driving was resumed. They knew that piles received support from end bearing and from skin friction, but they did not know, with certainty, what proportion of resistance arises from each pile, nor what changes in these proportions occur after rest and under load. These changes follow the slow return to equilibrium of the materials displaced by driving and the slow adjustment of increased water content pressures within the contracted voids in the disturbed and loaded areas.

The important point which was not stressed early enough was that it added nothing to the supporting power of a pile foundation to drive additional piles after the area affected had had inserted in it a sufficient number of piles to produce soil settlement from superimposed loads. Numerous examples of settlement trouble can be traced to this cause.

The second point was a lack of appreciation that under certain conditions the soil surrounding a pile from which support was expected, in cases where such soil was compacted or settled from other causes, would re-act through skin friction on the pile and overload it and carry it down. Ordinarily, settlements from this cause are entirely distinct from the superimposed load of the structures which the pile foundations are designed to support. They may be caused by compression and gradual settlement of new fill through which the piles are driven. One serious case arose from the placing of a heavy fill surrounding a concrete stadium which was properly enough designed, with sufficient concrete piles to carry, safely, the calculated loads from the structure, but these piles were unable to carry the added load arising from the settlement of soil compressed by the much greater load from a fill placed outside of, and around, the structure.

Somewhat more difficult to foresee was the settlement of batter piles driven to resist the outward movement of a quay wall constructed in 30 ft of water in a heavily silt-laden tidal stream. In this case the sheet-piles

for retaining the fill below and to the rear of the relieving platform were driven 40 ft back of the face of the wall, thus exposing the batter piles under the relieving platform to the accumulation of silt deposits from the stream. Routine and frequent dredging along the face of the wall would leave banks of soft mud under the wall held by the piles against rapid sloughing down into the stream. This added load from skin friction on the batter piles produced settlement of the piles, and instead of resisting the outward movement of the wall they actually assisted in pulling it forward so that years later, after the wall had moved out-board several feet and was being reconstructed for other reasons, excavation showed open joints between the batter-pile heads and caps, with about an inch of drift-pin exposed.

It is by pointing out cases in which current practices are faulty that their limitations can be better visualized and appreciated. More literature along such lines will be helpful. Much light can be thrown on this problem by engineers who are fortunate enough to combine with their work of design and construction other duties of maintenance, alteration, and repair of old structures. The exposure of parts incident to reconstruction provides an opportunity for examination of members subjected to the test of years and often reveals conditions, the study of which demonstrates the accuracy or error of past standard practices. It is the accumulation of such facts which gives an engineer a background of experience leading to safer construction, and inasmuch as specific concrete examples are more easily visualized and remembered than abstract rules, young engineers, by the perusal of such examples, can more quickly gather from the experience of others that knowledge of what to avoid and what limitations must be kept in mind which insures safe and permanent construction.

The examples cited in the paper and the lessons drawn therefrom are useful; they focus attention on the need for such subsurface exploration of soils around and below the piles as the importance of the structure warrants, and also the need for recording permanently such data for future use by others. In the case of less costly constructions where the expense of subsurface exploration is not warranted, they help to accentuate the importance of spreading pile spacings and analyzing possible future physical changes in surrounding conditions so as to avoid excessive or irregular settlements of soil in contact with, or below, the piles, and thereby assist in safe design.

The paper alludes to the possibility, in some cases, of securing savings through reduction in the number of piles where pile-points reach hardpan or rock. In any consideration of such savings careful attention should be given to the advisability of increasing the size of the pile-point in the end bearing and the possibility that such points may have been damaged in driving. It is also of interest in this connection to appreciate that foundation engineers know less than they should of the permissible increase in safe compression unit values and in end-bearing values for piles driven in firm soil where the pile particles are restrained against the lateral

expansion and disruption incident to the application of compressive loads. As this side restraint increases with the depth to which the piles penetrate, the safe unit load for compression and the resistance to crushing in wood and concrete must increase materially, depending, of course, on the character of the soil penetrated. Such increases in permissible safe compression are undoubtedly considerable in sandy or firm soil and possibly sufficient where piles have moderate tapers to permit the carrying capacity to be determined by the safe bearing on the fibers at the top of pile. Scientific investigation along this line would probably bring into closer agreement present practices in permissible loads on piles as compared with small caissons sunk to rock or hardpan bearing.

Present practices are conservative as far as piles driven to firm bearing strata are concerned. Not much trouble is encountered on pile foundations in sand or gravel where these materials are confined against possible future lateral movement. It is the compressible silts, mud, clay, and peat strata either below the piles or surrounding them, which should be investigated more thoroughly and studied cautiously.

Following thorough subsurface exploration much can be accomplished in certain soil combinations through the more scientific study of soil reactions and in the placing of piles to take economic advantage, safely, of any favorable factors which may be present. By securing a better distribution of loads to soil through a greater use of batter-piles, by lagging piles to secure controlled lengths and greater bearing at predetermined elevations, and, in certain cases involving lateral reactions, a more intelligent use of tension piles, a closer approach to the more exact methods used for the design of superstructures may be secured. A case illustrative of possible savings through use of lagging on piles exists at the U. S. Training Base on San Clemente Island, in California, where it is desired to extend a steel-pile pier to deeper water. The piles pass through successive layers of softer rock and sandy soil before hard rock is encountered. If carried to bed-rock, the steel piles at the outer end would be long and heavy; still it would not be safe to stop the narrow steel-pile points in any of the upper, softer, rock strata. Test piles show the strata to be fairly uniform and dipping toward the sea, and it is quite feasible to lag the piles with wooden fillers and secure safe bearing on one of the higher layers of softer rock. Somewhat similar would be the conditions encountered in coral where carefully planned use of lagging might reduce to reasonable figures what otherwise would be prohibitive costs of construction. In softer soils lagging, under certain conditions, would avoid the use of long composite piles. Such lagging, however, is feasible only for cases such as in pier construction, etc., where piles are single and not closely spaced in clusters, and not where the piles without lagging carry all the load the soil would support. In other cases, lagging will increase the effectiveness and value of piles subjected to tension.

The attainment of the objective of improving pile foundation design practice so as to bring it closer to the more exact methods used for super-

structure design, however, is being retarded by the uncertainties surrounding and affecting the interpretation of standard pile-driving formulas which seldom take into consideration such vital and important factors as the thickness of, or material used for, cushions, the effect of jetting, speed of driving, and vibrations from hammer or soil irregularities, and from methods of holding a pile, etc. In time, some of these uncertain elements will be better investigated and better understood. A simple contrivance developed by Commander W. Mack Angas (CEC), U. S. N., M. Am. Soc. C. E. (used experimentally in the driving of a 20 by 20-in. concrete test pile for a Navy pier at the Supply Depot in San Diego, Calif., in 1927), secured such remarkably accurate data as to promise definite progress in the investigation of certain factors.

This apparatus consisted of a revolving cylinder, 15 in. in diameter and 18 in. high, mounted on a platform adjacent to the concrete test pile being driven, and a pencil inserted in a socket near the head of the pile and bearing against a small spring at the base of the socket. This pencil traced a line on a sheet of paper mounted on the revolving cylinder surface, with the result that a record was secured of the exact movement of the head of the pile for each successive blow of the hammer. Time increments could be read to less than one-hundredth part of a second. With a similar cylinder, 6 ft long, the exact behavior and velocities and rebound of the steam hammer were recorded and could be read in equally fine time increments for all parts of the stroke. The graphs secured showed clearly the considerable rebound of the concrete pile-head to the shock of the hammer blow and, by proving the temporary occurrence of tension in the pile, explained certain horizontal cracks in piles driven through hard coral on a previous pier constructed at Pearl Harbor, Hawaii. It was the intention to use the apparatus in the study of the effect of variable cushions commonly used in pile-driving apparatus, but this was prevented by lack of time, money, and opportunity for further experimentation. However, the possibility of accurate analysis was established, and some day this interesting and important element will be investigated further. Any company or engineer in a position to carry this study further, either in analyzing hammer actions or efficiencies or pile behaviors can undoubtedly secure from the Bureau of Yards and Docks, U. S. Navy Department, Washington, D. C., details of the apparatus, and information and charts covering the data secured in San Diego in 1927.[7]

CHANDLER DAVIS,[8] M. AM. SOC. C. E. (by letter).[8a]—Quite properly, Mr. Miller warns designing engineers not to depend on "some convenient tabulation of friction values." A thorough study of the underlying soils is required. This is especially true in New York Harbor, where the most varying and peculiar conditions are encountered.

The late G. S. Greene, Jr., M. Am. Soc. C. E., Engineer-in-Chief of the Dock Department of the City of New York, confronted with these

[7] *Bulletin 36*, Public Works, U. S. Navy Dept.

[8] Cons. Engr., New York, N. Y.

[8a] Received by the Secretary September 22, 1937.

difficulties, designed his well known floating wall.[9] However, each authorized section was thoroughly studied before work was begun and the final design of the quay wall determined. Table 4 contains some of the observations made over the area of the West 23d Street, North River Section. This section extends from 10 ft south of the north side of West 23d Street to the north side of West 30th Street, about 1 944 ft.

TABLE 4.—CHARACTER OF BOTTOM AS DEVELOPED OVER THE WEST TWENTY-THIRD STRRET SECTION OF THE NORTH RIVER, AT NEW YORK, N. Y.

Location	AVERAGE DEPTH, IN FEET, BELOW MEAN LOW WATER								
	Water		Dock Mud		Compact Mud		Blue Clay*		Rock Bottom
	From:	To:	From:	To:	From:	To:	From:	To:	At
South, 1 238.2 ft..........	0	12	12	64	64	148	148	168	168
North, 615 142.0 ft........	0	16	16	76	76	144	144	153	153

* With gravel, sand, and shells.

Table 5(*a*) shows the penetration per blow at various depths at the foot of West 26th and 27th Streets, North River. In this case all the piles were straight, free of bark, and equipped with a cast-iron shoe. Blows were struck with a 1.5-ton hammer, falling 8 ft.

As shown in Table 5(*b*), the piles tested are part of the completed pier at the foot of West 26th Street, North River. At the time of the observations the pier had been completed about one year. All the piles tested were in the interior, each supporting about 16 tons; they were about 75 ft long, and the points were about 65 ft below mean low water. According to Table 5(*a*) the penetration per blow at this depth is about 6 in. and, by the *Engineering News* formula, the safe load is 3 tons per pile. The piles, however, were loaded to 35 tons per pile without damage to the pier. They are true friction piles and depend on the skin friction to support the working loads.

Whenever a large ship is breasted into the slip around an outer corner a perceptible movement of the pier is observed. The piles move and bend under the pressure of the vessel, acting as long columns of indefinite length. About 7 ft of their upper ends are braced securely and are fixed both laterally of the pier as well as longitudinally, and they act as one unit. At which point below the mud-line the lower ends are held immovable is indeterminate. The distance varies; each time the pier moves the mud grip is broken and the column length is increased by an unknown amount. However, as time goes by, the river bottom under the pier is built up; material in suspension in the river water is gradually deposited; the increased weight on the dock mud compresses it and renders

[9] See reference to paper by the late S. W. Hoag, M. Am. Soc. C. E., based on a statement in American Civil Engineers Pocket Book, Second Edition, 1913, p. 608.

it more dense, and thus the area over which the skin friction acts is increased and the pier becomes more stable. The writer removed a pier at

TABLE 5.—DRIVING DATA, PILES AT THE FOOT OF WEST TWENTY-SIXTH STREET AND WEST TWENTY-SEVENTH STREET, NORTH RIVER, AT NEW YORK, N. Y.

Pile No.	(a) COMPUTED* SAFE LOAD, PILES AT THE FOOT OF WEST 26TH STREET AND WEST 27TH STREET						(b) BEARING VALUE OF PILES NOT DRIVEN TO "REFUSAL"; NEW PIER No. 56, AT THE FOOT OF WEST 26TH STREET, NORTH RIVER, AT NEW YORK, N. Y.					
	Penetrations, in Inches per Blow, at the Following Depths, of the Point, in Feet, Below Mean Low Water:						Diameters, in Inches		Pile in Contact with Mud		Load per pile, in pounds	Resistance in mud, in pounds per square foot
	60	65	70	75	80	85	Head†	At the mud line	Surface area, in square feet	Depth, in feet		
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
1	1.0	..	..	..	..	..	13.87	10.10	83.97	39.91	59 900	714
2	0.7	..	..	..	..	..	16.00	11.03	85.60	38.51	59 900	699
3	1.0	0.6	0.6	0.5	0.4	0.3	13.25	9.67	80.37	39.20	67 500	839
4	0.6	0.6	0.6	0.5	..	..	13.75	9.96	83.37	39.91	68 075	816
5	1.1	0.8	0.7	0.6	..	..	14.13‡	11.00	88.50	36.31	67 500	763
6	1.3	1.0	..	..	..	..	13.25	9.81	85.45	41.31	70 550	826
7	0.7	0.5	0.6	0.3	..	..	12.25	9.34	87.50	43.61	68 075	778
8	1.7	0.6	0.6	0.4	0.4	0.3	16.50	11.62	96.82	42.00	70 550	729
9	..	..	..	..	..	..	15.25	10.65	86.95	39.97	70 550	811
10	..	..	..	..	..	..	15.50‡	11.83	97.30	37.51	70 550	725
11	..	..	..	..	..	..	16.75	11.70	93.37	40.51	70 550	755
12	..	..	..	..	..	..	14.75	10.72	93.88	42.91	70 550	751
13	..	..	..	..	..	..	12.62	9.50	82.98	40.91	68 125	821
14	..	..	..	..	..	..	16.00	11.46	95.32	41.71	67 500	708
Average	0.9	0.7	0.6	0.5	0.4	0.3	....	....	....	....	....	767§
Average safe load, in tons*	2.0	2.6	3.0	3.4	4.0	5.2	....	....	....	....	....	....

* Computed by the *Engineering-News* formula.
† Diameter of point is uniformly 6 in., except as noted.
‡ Diameter of point, 8 in.
§ Point resistance included.

the foot of Broad Street, East River, where the surface of the mud under the pier was level with mean low water. This deposit was of a very stiff consistency.

The foregoing are a few of the numerous observations made in New York Harbor on piles, all of which were not driven to refusal but are supporting their working loads. The pile-driving formulas are not applicable in these cases.

The observations and study of the river bottom in New York Harbor made by the Dock Department, covers more than 65 yr. The thorough study made by Mr. Miller of soil reactions in many countries bears out the foregoing findings and deductions. He further stresses the necessity of a thorough investigation of the subsoil and a complete understanding of the facts obtained; that is, Mr. Miller emphasizes the fact that experience is an important factor in such studies, particularly in the case of piles that are not driven to " refusal ", and are friction piles.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

EARTHQUAKE RESISTANCE OF ELEVATED WATER-TANKS

Discussion

BY MESSRS. N. H. HECK, MASON A. STONE, AND H. C. BOARDMAN

N. H. HECK,[11] M. AM. SOC. C. E. (by letter).[11a]—During recent years attention has been given to earthquake-resistance design of structures. In several places these investigations have been supplemented and based largely upon the results of tests of models of the structure on a shaking platform. At the Massachusetts Institute of Technology, at Cambridge, Mass., a light model can be tested on a platform which may be put into any desired kind of motion, including the reproduction of actual earth motion.

As indicated in Professor Ruge's paper the U. S. Coast and Geodetic Survey has not only made it possible to reproduce the earthquake motion through double integration of the acceleration records of the Long Beach (Calif.) earthquake, obtained at Los Angeles, but it has determined the periods of many tanks of different types and designs, thereby adding greatly to the validity of his conclusions. It will soon be possible to know the degree of accuracy of the deduced earthquake motions through shaking platform tests of the accelerometers made in May, 1937. Although the records have not yet been studied the results appear promising.

In outlining the special problem Professor Ruge has performed a service which extends to a wider field in showing that the earthquake manifests itself as inexorable motion rather than as direct force, and that the forces exerted depend primarily on the characteristics of the structures undergoing the motion. The complexity of his apparently simple problem becomes obvious when pointed out and indicates the important place taken by tests of a suitably designed model on a shaking platform.

The validity of the term, "artificial earthquake motion" (see heading, "Artificial Earthquake Motion"), may be questioned. Apparently, Pro-

NOTE.—The paper by Arthur C. Ruge, Assoc. M. Am. Soc. C. E., was published in May, 1937, *Proceedings*. This discussion is printed in *Proceedings*, in order that the views expressed may be brought before all members for further discussion of the paper.

[11] Chf., Div. of Terrestrial Magnetism and Seismology, U. S. Coast and Geodetic Survey, Washington, D. C.

[11a] Received by the Secretary June 4, 1937.

fessor Ruge has in mind that no earthquake record is yet available that contains the wide range of so-called destructive periods, so he uses the best substitute therefor. When supplemented by the use of earthquake motions actually observed, this procedure is the best available at present.

It should be understood that the Coast and Geodetic Survey has not yet found it possible to integrate all the strong-motion accelerograms that it has, and until this has been done the record cannot be applied to controlling the motion of a shaking platform. Lack of personnel has prevented this work and a special effort is being made to find whether the records can be integrated by the differential analyzer at the Massachusetts Institute of Technology. Furthermore, Frank Neumann, Seismologist, U. S. Coast and Geodetic Survey, is developing a torsion pendulum analyzer which deals with the specific problem. Attention may be called to the fact that although all the strong-motion accelerometers in California are not able to record the greatest possible range of earth motion, twelve instruments are equipped with a double-magnification system so that if the instrument survives a record will be obtained even if the acceleration is 0.5 g, or more.

In determining the vibration periods and in recording earthquakes in actual structures, the Coast and Geodetic Survey is concerned with design, and is anxious that every possible application be made of the information that it has obtained.[12]

Nevertheless, it must be remembered that it is not engaged in structural design. In some respects this may be inconvenient for the engineer, but on the other hand it has the advantage that results are obtained without any desire to support any particular theories, and this may prove of great advantage in the future. Accordingly, the writer will not discuss Professor Ruge's proposed design further than to state that the Long Beach earthquake proved the inadequacy of present design to resist severe earthquakes and that these structures will have to be designed in some other way in order to survive.

No exception is taken to Professor Ruge's remarks as to the element of chance (see heading, "Element of Chance"), except to note that he fails to emphasize one element that may be the key to the great variety of observed effects—the varying conditions of the geological substratum. This can cause changes in the periods and amplitude of the motion and the amount of energy. However, this merely emphasizes the expressed view that valid conclusions can be based only on a large number of cases.

It should be emphasized that the records of the Long Beach earthquake, at Long Beach, and at Vernon, Calif., and the records of the Lower California earthquake of December 30, 1934, at El Centro, contain valuable information which could be revealed by integration, since each record represents motion of a unique geological formation.

MASON A. STONE,[13] ESQ. (by letter).[13a]—Being of interest not only to the limited number of designers who specialize in the elevated water tank,

[12] "Earthquake Investigations in California, 1934–35," *Special Publication No. 201*, U. S. Coast and Geodetic Survey.

[13] Structural and Mech. Engr., New York, N. Y.

[13a] Received by the Secretary September 7, 1937.

but to those who are called upon to design steel structures carrying heavy loads at the top, Mr. Ruge's paper, of course, has a broader application than the title indicates. Among these steel structures, for instance, are viaducts with a heavy deck traffic, towers in chemical plants carrying tanks or machinery, elevated storage bins, hoisting towers with heavy sheaves or hoisting engines at their tops, etc.

The water tower, or rather the tower carrying the elevated tank, as the author shows, is further complicated by the action of the water surges. Therefore, it is necessary to consider as superimposed on the natural vibration period of the tower the period and amplitude of the motion of the ground due to the earthquake and the period and momentum of the water. A rational method of designing the structure would require the evaluation of the stresses set up by these three motions.

Whether or not such a mathematical analysis is possible, it is highly desirable that the conclusion be checked by experiments with actual models, and it is fortunate, therefore, that the apparatus for conducting such tests is available and that experiments with such models are actually being conducted. This paper must be considered as a progress report of such experiments. It is valuable: (1) Because it demonstrates the errors inherent in designing from the simple theory based on a constant acceleration; (2) because it indicates the difficulties and expense of attempting to meet the problem by increasing the stiffness of the tower; (3) because it suggests a mechanical solution whereby the structure, within certain limits of height of tower and capacity of tank, may be made flexible enough to absorb the stresses; and (4) because it reveals the tendency of the tower to continue vibrating in resonance with the earthquake shock, or after the earthquake shock has ceased, and shows how such vibrations may be damped.

The data given will enable existing towers to be examined for earthquake-resistant properties and, within the limits given, to be re-designed by the addition of the springs and dampening device so that they may be safe. However, the paper affords no general solution for the problem of designing the elevated tank tower. The stresses in the tower are the result of the amplitude of the vibration at the top, caused by the period and amplitude of the earthquake motion, and modified by the momentum of the water in the tank.

Although the stiffness of the tower may be computed, its period of vibration depends on the initial stresses set up in the members during the erection and the stresses due to the weight of water in the tank. Fortunately, reports are available containing data on the stiffness and period of vibration of a number of tank structures. In this connection, however, it may be remarked that the type of tower selected for the experiment and designated as the standard design is a very poor one from the standpoint of earthquake resistance. Apparently, the batter of the legs was considered solely from the standpoint of resisting the overturning moment due to wind. If one is to design for earthquake stresses a much broader base for the tower would seem to be indicated. On the other hand, for a struc-

ture of uniform strength to resist a horizontal force at the top a parabolic curve concave on the outside is indicated. This form is approximated in the design of the towers for the support of high-tension transmission lines. The tower with a wider base also recommends itself in connection with the increasing foundation loads due to earthquakes.

There is a considerable record of the magnitude and velocity of earthquake motion, but it is not readily accessible to the ordinary structural designer as it is contained in the reports of the Imperial Japanese Earthquake Commission and the Seismological Society of America which are not to be found in the average library. It is regrettable that Mr. Ruge did not include some statement of the period and amplitude which might be expected in earthquake motions. His statement that there was only a record of one earthquake available for study (that is, for cutting the necessary optical cam) might have been amplified by the explanation that the ordinary seismograph does not give the necessary information; an accelogram is required.

In the tank under consideration, the action of the water is shown to have a dampening effect upon the vibration of the structure, but this might not be true for all proportions of tanks and towers. It would seem a profitable line of investigation to determine the period and forces set up in water tanks of various proportions and capacity, at least for simple harmonic motions. At present, the proportions of the tank are usually determined by the most economical use of the material.

In this connection it may be suggested that waves set up by using only one component of the earthquake motion at a time may not be representative of the actual action of the water during an earthquake. Undoubtedly, the actual motion of the tank is roughly elliptical. Instead of being reciprocal, therefore, the wave would have a circular motion around the tank; it might have a different period and might develop different forces from those determined in experiments using only a reciprocal motion of the shaking-table.

When more is known about the action of the water, it may be possible to determine optimum proportions of tank and tower for the minimum vibration. The possible effects of the riser pipe also appear to merit consideration. Of course, if the pipe is secured at the bottom of the tank and to a foundation in the ground, and if it is also tied to the tower by horizontal braces at the panel points, a vertical column of water will somewhat reduce the center of gravity of the system as a whole. Furthermore, the stiffness of the pipe will add to the stiffness of the structure, although not very materially as it is situated at its neutral axis. In this case the stresses in the connection at the bottom of the tank and the foot elbow on the foundation should be investigated; but the pipe itself might be expected to resist any deflection that the legs of the tower are called upon to take.

It has occurred to the writer, however, that if the pipe is provided with a universal joint at the top and swiveled at the bottom connection so that it may swing freely as a pendulum, its natural period will probably

be different from the structure as a whole and from that of the earthquake, and, therefore, it may have a beneficial dampening effect. This idea occurred to the Japanese when building pagodas. They introduced a heavy timber suspended from the top and appear to have had beneficial results with this type of construction.

The idea of designing a structure 100 ft high to withstand a deflection at the top of 15 in. is somewhat startling to a structural engineer. The secondary stresses at the connections between the horizontal struts and legs would have to be given careful consideration unless the author's suggestion of eliminating them by pin connecting the struts was adopted. Furthermore, the stresses in the riveted connection between the tank and the legs may possibly be relieved by the deflection of the tank sheets, but this action might result in the failure of rivets, or sheets.

The author's conclusions for the design of tanks and structures within the limits indicated appear to be valid. Of course, the damping device would have to be one that would not be affected by frost or that would not deteriorate with age and neglect.

H. C. Boardman,[14] M. Am. Soc. C. E. (by letter).[14a]—By his painstaking and well directed efforts, Mr. Ruge has developed useful information in a field where it is much needed. The following excerpts from his paper are especially significant:

(*a*) "The type of tower now in common use was not developed for earthquake resistance, * * * ; there is no reason to accept it as the ideal structure for withstanding earthquake motions, which actual experience and repeated laboratory tests have proved it certainly is not." (See heading, "Introduction: Nature of the Problem.")

(*b*) "Among the uneconomical designs are those which seek to give the structure adequate quake-resistance by providing an extremely stiff or rigid tower. * * * If the natural period cannot be reduced to about 0.5 sec, it is doubtful whether the tank is very much more likely to survive a violent earthquake than a standard design built for wind resistance only." (See following Table 1.)

(*c*) "The natural conclusion from a prolonged study was that, barring fantastic and impractical designs, the best way to obtain adequate quake-resistance is to provide the tower with a greater range of elasticity; that is, if the tower can be built so that the safe deflections are large enough, there will be no danger of failure." (See following Table 1.)

(*d*) "There are two general methods of increasing the allowable or elastic deflection range of a structure such as a water-tank tower: (1) By making the tower of some type of open framework in which the forces resisting deflection are produced by bending stresses set up in some or all of the members of the framework; and (2), by artificially increasing the allowable direct-stress deformations of some or all of the members of a statically determinate tower." (See following Table 1.)

Taken at its face value, Excerpt (*a*) suggests that towers of the conventional type be not used in earthquake zones. Apparently, the author

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[14] Research Engr., Chicago Bridge & Iron Company, Chicago, Ill.

[14a] Received by the Secretary September 18, 1937.

did not intend to convey this meaning. Nevertheless, it is well worth careful consideration, since other commercially and structurally satisfactory forms of towers can be designed.

Excerpt (*b*) states that a very stiff tower would safely resist earthquakes, but that such a tower would be uneconomical. In this connection, the type of structure illustrated by Fig. 27 is offered for consideration. It is stiff, but, nevertheless, offers promise of economy.

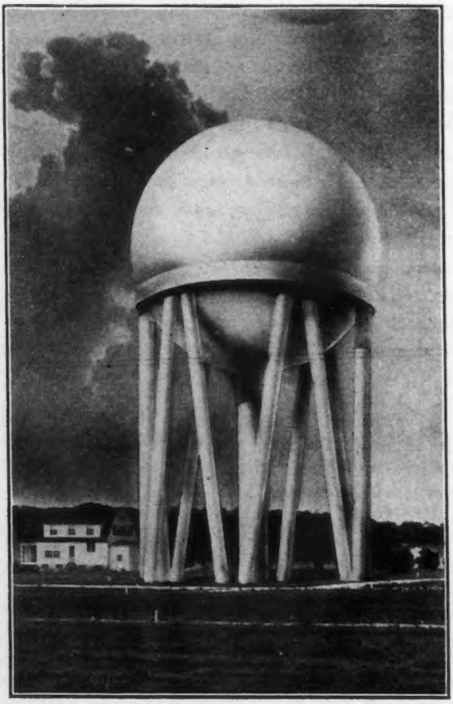

FIG. 27.

FIG. 28.

Excerpts (*c*) and (*d*) present the author's conclusion that the most practical method of providing earthquake protection is to make the tower flexible, either by use of an open framework with the members subjected to bending stresses, or by increasing the direct stress deformations of some or all of the members by the use of springs or similar devices. It is pertinent to re-emphasize what the author has stated, namely, that many tanks and towers of the conventional design have withstood earthquakes with little damage, except the breaking or stretching of some of the rods, which were easily replaced. Why, then, would it not be feasible to use longer panels and incline the rods more steeply, so that greater horizontal translations of the tower would be required to stretch the rods to the

yield point and through the plastic range? Fig. 28 is offered as an example of such a design. Over a period of years, springs would almost

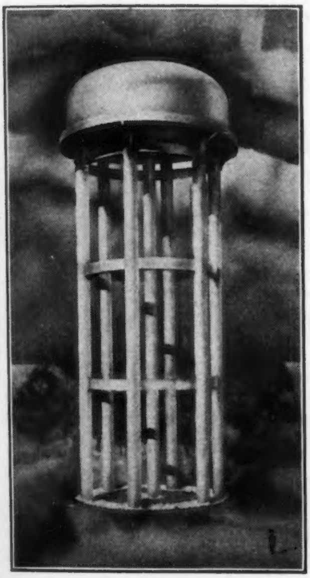

FIG. 29.

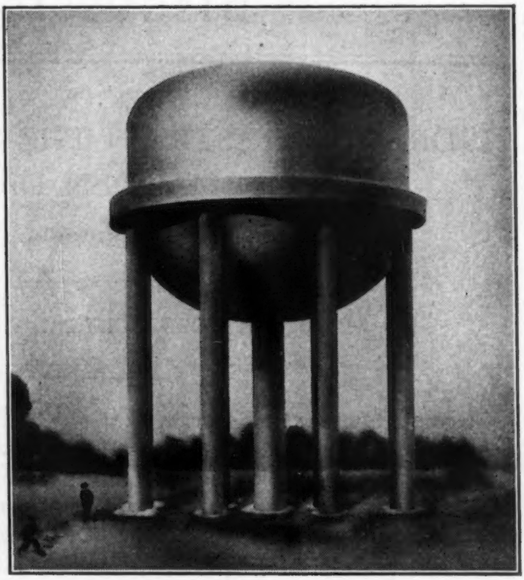

FIG. 30.

surely be subject to serious maintenance troubles, and, therefore, they should not be used if designs not requiring them can be developed. Figs. 29 and 30 illustrate designs of the open type, without diagonals or struts.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

HYDRAULIC TESTS ON THE SPILLWAY OF THE MADDEN DAM

Discussion

By Messrs. Ettore Scimemi, and J. C. Stevens

Ettore Scimemi,[10] Esq. (by letter).[10a]—Those who are studying the possibility of applying experimental results obtained from model tests to the prototype, will find this paper of the greatest interest. Experiments of the type demonstrated by the author are now very often made in laboratories, whereas those obtained directly from the original structure are very rare or are lacking entirely. Knowing how very difficult it is to make exact measurements, the writer feels that Mr. Randolph deserves unlimited praise. Especially interesting is the fact that there is good correspondence between the discharges and the distribution of the pressure along the crest of the dam.

For the shape of the spillway of the Madden Dam the author has adopted a circular profile; and yet this profile is neither the most economical nor the most suitable one for a spillway dam. One type that is now quite popular is that recommended by William P. Creager, M. Am. Soc. C. E., about 1917.[11] About 1930 the writer analyzed this profile,[12] but only with regard to the shape of the hydraulic sheet (see Fig. 22); he noted then that the Creager profile is always above the lower nappe of the sheet.

Mr. Creager gives the co-ordinates of this profile in a table which is now included in almost every handbook on the subject.[11] The zero, or the origin, of reference should be referred to the vertex, or the summit, of the dam, as shown in Fig. 23 and Table 2. It is not necessary to describe a complete curve for the up-stream part of this vertex. The lower nappe

Note.—The paper by Richard R. Randolph, Jr., Esq., was published in May, 1937, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion of the paper.

[10] Professor, Director, Hydr. Inst., Univ. of Padua, Padua, Italy.

[10a] Received by the Secretary July 23, 1937.

[11] "Masonry Dams", by William P. Creager, 1917, p. 108.

[12] "Sulla Forma delle Vene Tracimanti", *L'Energia Elettrica*, Milan, April, 1930.

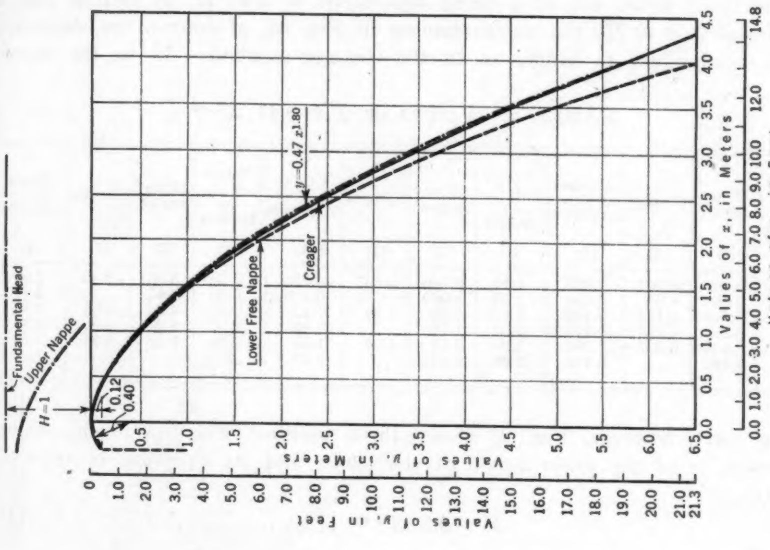


FIG. 23

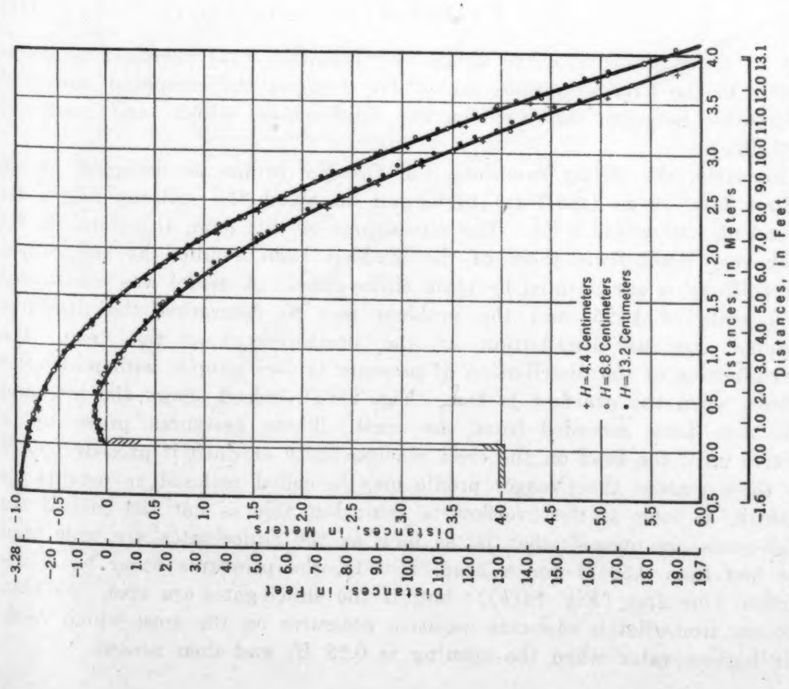


FIG. 22.—EXPERIMENTAL SHEET, UPPER AND LOWER NAPPE

adapts itself to an arc of a circle equivalent to 0.40 H, in radius, and a rise equal to 0.12 H; the measurements in Fig. 23, of course, are computed for a head equal to unity, as in the Creager method. As to the down-

TABLE 2.—VALUES OF $Y = 0.47\ X^{1.80}$

Values of X, in meters (1)	Free nappe (2)	Creager (3)	Values of $Y = 0.47\ X^{1.80}$ (4)	Values of X, in meters (1)	Free nappe (2)	Creager (3)	Values of $Y = 0.47\ X^{1.80}$ (4)	Values of X, in meters (1)	Free nappe (2)	Creager (3)	Values of $Y = 0.47\ X^{1.80}$ (4)
0.10	0.007	0.007	0.007	1.10	0.570	0.567	0.555	2.70	3.05	2.82	2.81
0.30	0.055	0.06	0.054	1.50	1.060	....	0.975	3.00	3.65	....	3.40
0.50	0.140	0.142	0.145	1.70	1.33	1.22	1.22	3.20	4.20	3.82	3.81
0.70	0.260	0.257	0.248	2.00	1.75	....	1.65	3.70	6.00	4.93	4.96
0.90	0.400	0.397	0.390	2.20	2.10	1.96	1.95	4.20	7.25	6.22	6.22
1.00	0.480	....	0.470	2.50	2.65	....	2.45	....	....	....	....

stream part, however, Fig. 23 shows three profiles: The lower one, which is dotted, is of the lower nappe of the sheet, and its equation is approximately,

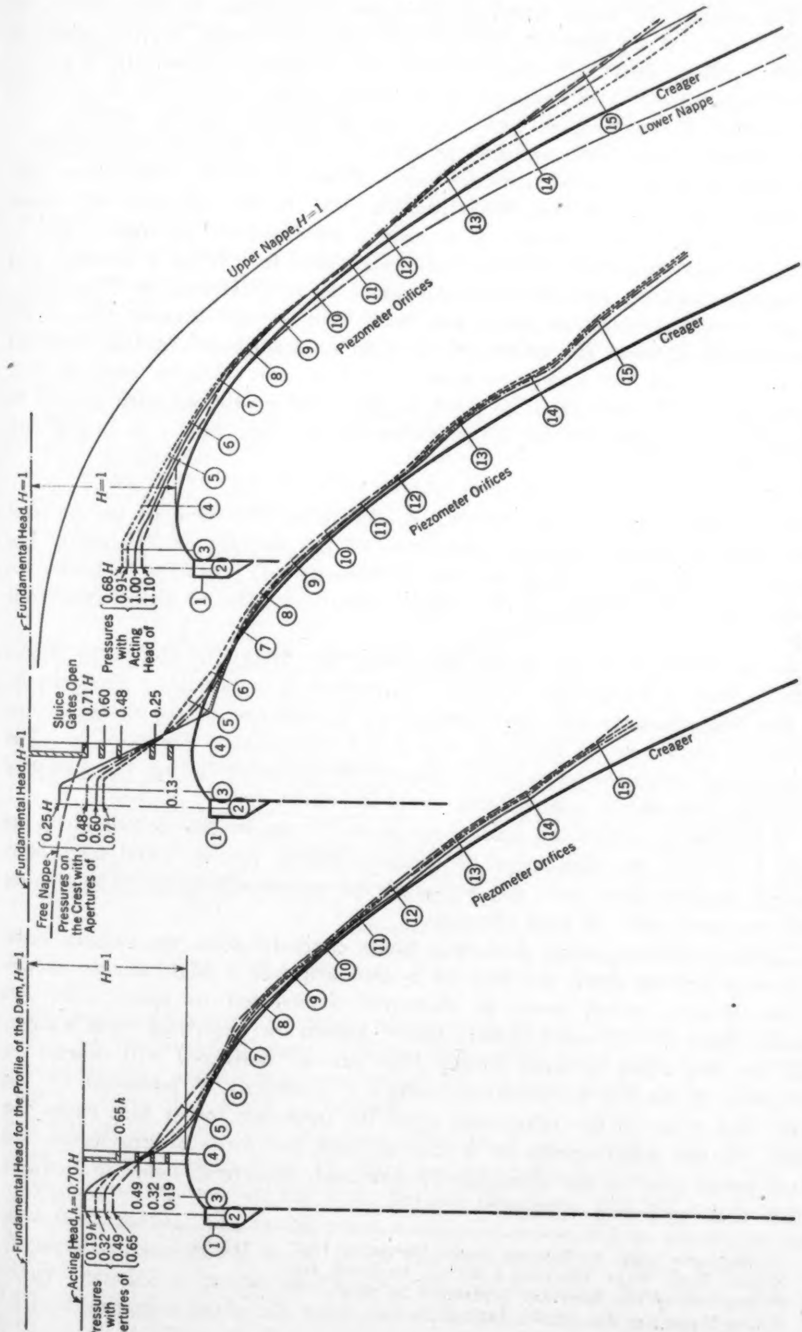
$$Y = 0.5\ X^{1.85} \qquad (11)$$

the upper one, a solid curve, is that given by Mr. Creager, and the third, identified by dots and dashes, is the curve of the equation,

$$Y = 0.47\ X^{1.8} \qquad (12)$$

Fig. 23 shows that the curve defined by Equation (12) corresponds almost exactly to the Creager profile, and Table 2 shows the computed numerical differences between the two curves (differences which are absolutely negligible).

Recently, the writer examined the Creager profile as designed for an Italian dam, 70 m (229.7 ft) high, and on which the spillway allows for a head of 4.40 m (14.4 ft). The dimensions of this dam, therefore, do not differ very much from those of the Madden Dam, studied by the author. The spillway is surmounted by plain sluice-gates. A model was constructed on a scale of 1:25, and the problem was to determine the discharge capacity and the distribution of the pressures along the crest. The determination of the distribution of pressure is very simple, with piezometer orifices, when the overflow is free. Fig. 24(a) indeed shows the pressures that have been recorded from the crest. These pressures prove to be positive until the head on the crest reaches unity or until it exceeds 1.10 H For these reasons the Creager profile may be called rational as regards the pressure, as long as the overflow is free; but this is not the case if the sluice-gates are opened; that is, as long as the sluice-gates are wide open (not less than 0.50 H for a head, $H = 1$), the pressures occur as if the overflow were free (Fig. 24(b)); but, if the sluice-gates are open less than 0.50, one immediately observes negative pressures on the crest which reach their highest value when the opening is 0.25 H, and then reverse.



Fundamental Head for the Profile of the Dam, H=1
Acting Head, h=0.70 H
Pressures with Apertures of
0.19 h
0.32
0.49
0.65
0.49
0.32
0.19
0.65 h
H=1
Piezometer Orifices
Creager
Fundamental Head, H=1
Free Nappe
Pressures on the Crest with Apertures of
0.25 H
0.48
0.60
0.71
0.13
Sluice Gates Open
0.71 H
0.60
0.48
0.25
Pressures with Acting Head of
0.68 H
0.91
1.00
1.10
Upper Nappe, H=1
Lower Nappe
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15

FIG. 24

The profiles of these negative pressures are shown in Fig. 24(*b*), in which, however, one can observe that this zone of vacuum, on the whole, is rather limited, and the absolute value of the vacuum is, generally, a matter of no importance. For instance, if the head on the dam is 10 m (32.8 ft) the vacuum is only 0.50 m (1.64 ft). In the greater number of cases one may assume that these negative pressures will not damage the structure, especially if it is a solid gravity dam. However, if it is a hollow type of dam, it is possible that the vibrations due to the vacuum will cause damage and then it is necessary to reduce the head on the dam. Mr. L. Escande[13] has suggested reducing the water head to 0.70 as a remedy, and the writer has been able to verify this value as is illustrated by Fig. 24(*c*), in which the head on the crest has been kept at not greater than 0.70; that is, 0.70 *H*, with the profile of the dam as described. In that case, all the pressures on the crest are positive; but it may also be seen in Fig. 24(*c*), that they come close to zero, so that the advantage with regard to the phenomena obtained by the experiments, on Fig. 24(*c*), is indeed not very remarkable.

J. C. STEVENS,[14] M. AM. SOC. C. E. (by letter).[14a]—One of the projects early adopted by the Special Committee of the Society on Hydraulic Research,[15] is to gather data on the conformity of prototype behavior to model tests. Mr. Randolph has added important data to the Committee's store.

In so many cases the works vary from the design developed by model tests, to such a degree that a direct comparison is impossible. Fortunately, in this case, the prototype was constructed to conform to the design shown by model studies to be the best, and direct comparisons are possible. The construction management is to be complimented for installing the facilities by which such direct comparisons could be made.

It should be recognized that there is no virtue in the sloping deck, as such, to aid in the dissipation of energy. Better results would have been secured had the deck been horizontal at the proper elevation. The sloping deck was used only to save concrete.

Little is known about hydraulic-jump characteristics on sloping beds. A stream flowing down the face of a dam acquires a high kinetic energy at the bottom, which must be destroyed (converted to heat). The hydraulic jump is the most potent factor known in absorbing such energy, and the proportion of total energy that can be destroyed will depend on the ratio of kinetic to potential energy ("kineticity") possessed by the water just prior to the jump and upon the opposing forces that cause the jump. If the jump occurs on a sloping deck the force of gravity on the water prism acts in the direction of flow and, therefore, tends to increase rather than to retard velocity.

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[13] "Barrages", par L. Escande, Paris, Hermann, 1937, p. III–66.

[14] Cons. Hydr. Engr. (Stevens & Koon), Portland, Ore.

[14a] Received by the Secretary September 20, 1937.

[15] Year Book, Am. Soc. C. E., 1937, p. 8.

In many cases, baffles of various types are placed on the apron to constitute an opposing force and thus aid the formation of the jump and insure its forming on the deck or apron. In this case, the deflector at the end of the apron serves as a retarding force and, to some extent, counteracts the force of gravity on the sloping deck. For high dams and low discharges per unit width, the force of gravity on a sloping apron may be relatively unimportant, but for greater discharges per unit width this force cannot be ignored.

The writer seriously questions the author's Conclusion (2) that the jump can be "controlled best by placing a sloping apron down stream, * * *." This conclusion cannot be supported in theory because little is known of the jump on such slopes. It cannot be supported in practice

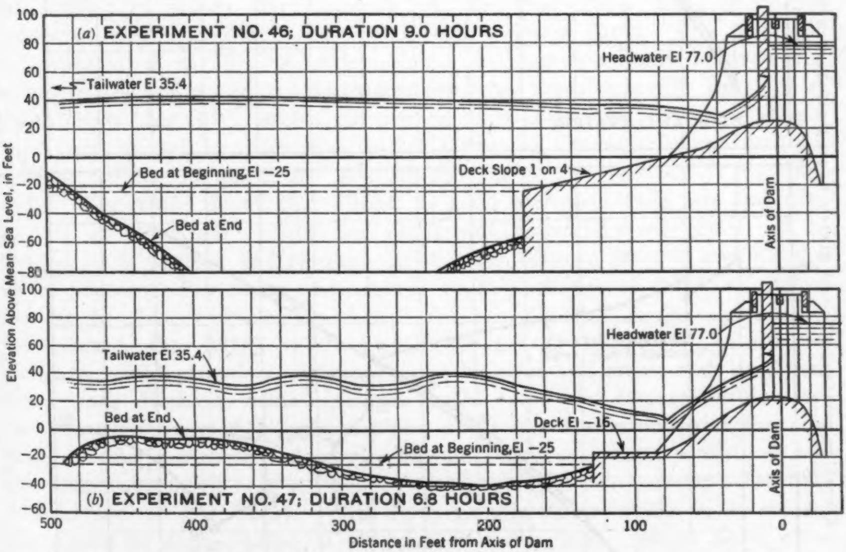

FIG. 25.—COMPARATIVE STUDY OF SCOUR, BONNEVILLE DAM, WITH A DISCHARGE OF 400 000 CUBIC FEET PER SECOND THROUGH EIGHT GATES

because the prototype discharge concentrations are too low to serve as a check on the model studies. For the Bonneville Dam with high discharge concentrations model studies showed the sloping deck to be the worst possible form that could be devised.

Fig. 25(*a*) shows the scour resulting from Experiment No. 46, with a deck sloping 1 on 4 from an elevation corresponding to — 25. The discharge concentration on the deck, beyond the piers, was 852 cu ft per sec per ft of width. The model river material was scoured to the flume bed—a depth corresponding to 50 ft on the prototype. If deeper material had been available it might easily have scoured to twice that depth. Of the 172 experiments made for scour prevention for this dam, Experiment No.

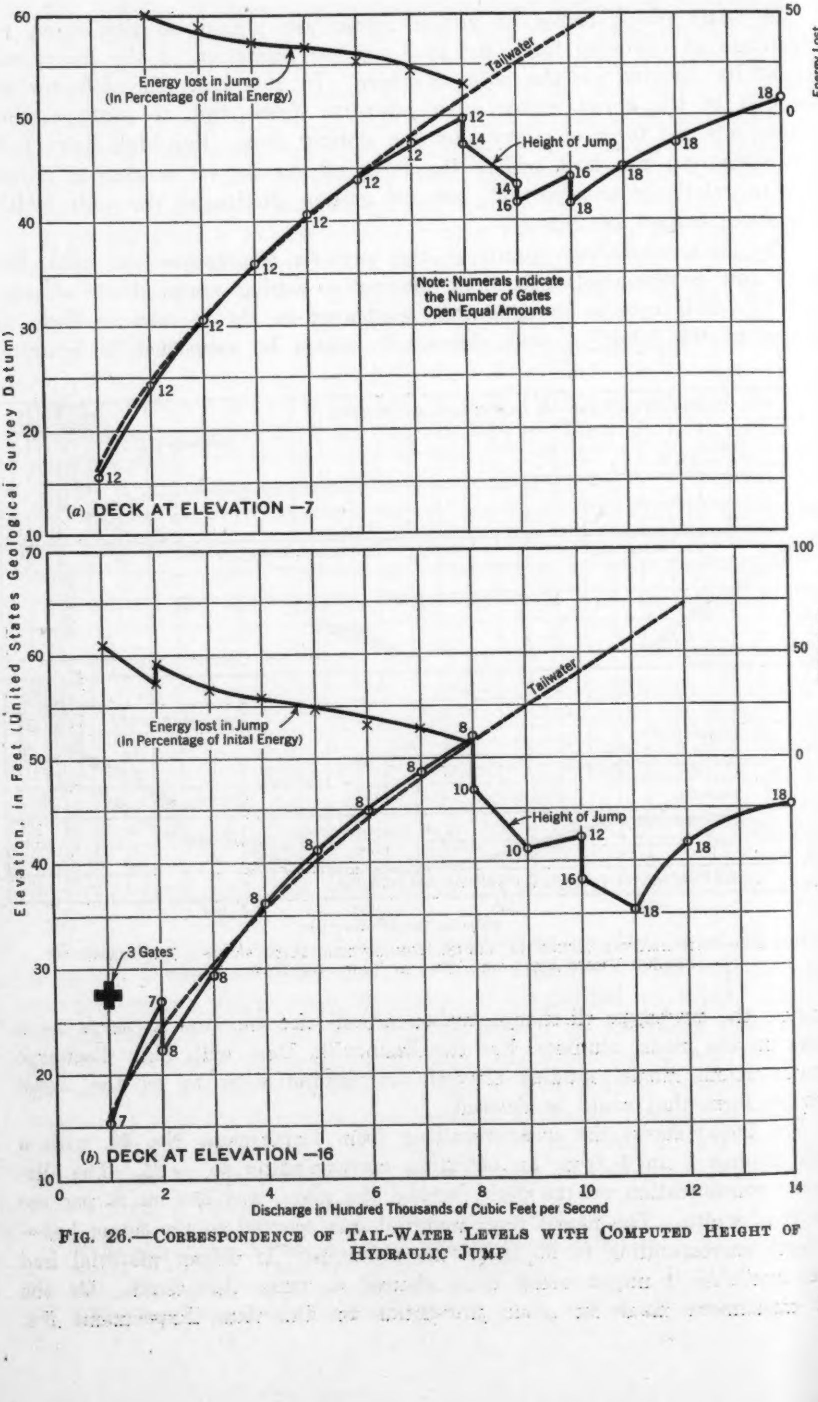

FIG. 26.—CORRESPONDENCE OF TAIL-WATER LEVELS WITH COMPUTED HEIGHT OF HYDRAULIC JUMP

46, with the 1 on 4 sloping deck, showed the maximum scour. Fig. 25(*b*) shows the results of Experiment No. 47 for the same discharge concentration, but with a horizontal deck corresponding to Elevation —16.0. The maximum scour corresponded to a depth of 15 ft. Although the length of run for Experiment No. 46 corresponded to 9 hr while that for Experiment No. 47 corresponded to 6.8 hr, the flume bed of Experiment No. 46 had been exposed completely and most of the scour had occurred in the first 7 hr. On the Bonneville deck it was necessary to place baffles[16] to insure the jump forming on the deck or apron and to aid in dissipating the energy.

Fig. 3 shows that if a horizontal apron could have been built at Elevation 83 a good correspondence between height of jump and the normal tail-water level could have been secured for practically the entire range of discharge. By proper manipulation of the crest gates as good correspondence might also have been had with a lower apron. This point is well illustrated in Fig. 26 which shows the correspondence between the height of the jump and the tail-water level for the Bonneville Dam. Fig. 26(*a*) shows that with twelve gates open equal distances, the deck should have been placed at Elevation —7.0 for good correspondence, for discharges to 600 000 cu ft per sec, or 850 cu ft per sec per ft of deck width. Fig. 26(*b*) shows that almost as good correspondence was had with a deck at Elevation —16.0 for discharges through seven gates up to 200 000 cu ft per sec and through eight gates between 200 000 and 600 000 cu ft per sec. The latter corresponds to a discharge of 1 275 cu ft per sec per ft of deck width.

Although Fig. 26(*b*) shows that discharges of 100 000 cu ft per sec per gate would give fair correspondence between tail-water and height of jump, such discharge concentrations would raise the forebay level above the allowable limit of Elevation 82.5. It is also to be observed that the ratio of kinetic to potential energy becomes low under such conditions and the jump can absorb only a small proportion of the total energy—less than 10% as seen in Fig. 26(*b*). With flows greater than 800 000 cu ft per sec the degree of submergence is so great that the jump cannot form effectively, and little energy is absorbed by it regardless of the number of gates opened.

An interesting feature of the prototype performance was obtained by the Pitot tubes projecting from little piers on the apron. From Fig. 16, it appears that the maximum velocity on the apron, just prior to the jump, was only about 69% of the theoretical in the prototype, whereas, in the model, it was about 91% for the same discharge. This variation needs more of an explanation than the mere difference in roughness of metal and concrete.

[16] See *Engineering News-Record*, January 14, 1937, p. 61, for description of the baffles used.

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DISCUSSIONS

ESSENTIAL CONSIDERATIONS IN THE STABILIZATION OF SOIL

Discussion

By Jacob Feld, M. Am. Soc. C. E.

Jacob Feld,[5] M. Am. Soc. C. E. (by letter).[5a]—A fine summary of soil stabilization is presented in this paper which covers especially the effect of the colloids and moisture films. The real complexity of the subject can only be visualized by a complete definition of the word, "soil," as used in the engineering sense. Such a definition does not yet exist, and an agreement on the meaning of the word would be a great aid in the classification of materials and tests concerning soils. In 1924, the writer[6] enumerated the major constituents of soils and their respective contributions to soil properties as well as the effect on the colloidal constituents of various chemicals and electrolytes.

The farm technologist defines a soil as a porous mass of hard framework plastered over with a jelly containing chemically active matter, plant foods, and unstable organic compounds, the pores of which contain air and water. The farmer classes soils as heavy (clayey) or light (sandy), referring to the predominance of the mineral constituent.

The geologist distinguishes between (1) primitive soils formed by the mechanical loosening and chemical decomposition of rocks (or a combination of both); (2) derived soils formed by the mixture of primitive soils with possible additional weathering, due to water or wind acting as collecting agents; and (3) decayed vegetation mixed with small percentages of minerals forming humus, peat bogs, or similar highly organic soils.

It is quite well agreed that studies on pure sand, pure clay, uniformly grained granular substances, powdered rocks, or any laboratory combina-

Note.—The paper by C. A. Hogentogler, Assoc. M. Am. Soc. C. E., and E. A. Willis, Esq., was published in June, 1937, *Proceedings*. This discussion is printed in *Proceedings*, in order that the views expressed may be brought before all members for further discussion of the paper.

[5] Cons. Engr., New York, N. Y.

[5a] Received by the Secretary September 3, 1937.

[6] Progress Report of the Special Committee to Codify Present Practice on the Bearing Value of Soils for Foundations, etc.: Discussion by Jacob Feld, M. Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., January, 1924, p. 114.

tions of these items, with or without water, can help but little, if any, in the solution of a field problem; and the impossibility of reproducing temperature changes, moisture distribution, and other variables, to say nothing of the difficulty of obtaining true soil samples, makes tests on actual soil samples of doubtful value. The foregoing statement would be true even in a uniform soil (a hypothetical and probably non-existent material).

Unfortunately, the problem has been attacked from the microscopic instead of the macroscopic point of view. It is so easy to study the leaves without even becoming aware of the existence of the tree. In the writer's opinion, much more valuable, as well as more rapidly usable, results will be obtained when an agreement is reached to classify soils on the basis of a few quantitatively measured physical properties, found by tests as simple as those now used for testing cement. Standards can then be determined for certain well known classes of soils, such as the list of "Important American Soils" issued by the Department of Agriculture.[7] Without such an agreement, tests on soils made by different men can never be compared, and the accumulation of test data will only be used by those who desire to show agreement between some chosen isolated tests and a "pet" theory.

[7] Year Book, U. S. Dept. of Agriculture, 1911, pp. 223–236.

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DISCUSSIONS

WATER TRANSPORTATION VERSUS RAIL TRANSPORTATION
A SYMPOSIUM

Discussion

By George Hartley, Esq.

George Hartley,[9] Esq. (by letter).[9a]—An important factor mentioned in the introduction of the paper (see "Historical") was excluded from the remainder of the text.

The author calls attention to the fact that, in the past, those who have wished to secure improvements for navigation on the Mississippi and Missouri Rivers have been aided greatly in securing Federal appropriations for river works: (*a*) Because great flood damage focused national attention on river improvement from an entirely different viewpoint; and (*b*) because "there was a desire to stabilize the banks which were quite subject to erosion * * *." No further references to either flood control or erosion are made in the paper, and, in a later statement, when evaluating the cost of capital improvements, no specific mention is made as to what proportion of the capital improvements may be assigned to flood control and erosion. For example, under the heading, "Existing Water Transportation Facilities: Routes," the author states: "The total expenditures chargeable to capital improvements for navigation on these main routes amounts roughly to about $550 000 000 to date (1937)." The specific reference to navigation at once infers, by reference to Major Putnam's previous statement, that some allowance may have been made in an attempt to proportion specific charges between navigation and the two factors, flood control and erosion. In the event that the total cost, as given, includes the expenditure for flood control and erosion, the capital cost utilized as a comparative economic study later in the paper could very well be reduced by an amount equal to the cost of these improvements, and the resulting

Note.—The Symposium on Water Transportation Versus Rail Transportation was published in September, 1937, *Proceedings*. This discussion is printed in *Proceedings*, in order that the views expressed may be brought before all members for further discussion on the Symposium.

[9] With Robinson & Steinman, Cons. Engrs., New York, N. Y.

[9a] Received by the Secretary September 18, 1937.

costs would place the waterway at once in a much more favorable position when viewed from an economic standpoint relative to railroad transportation.

No attempt has been made to evaluate the capital charge as well as the operation charges on bridges erected over these rivers to correct for increased cost of construction due to navigation requirements. Mr. Wonson makes the following statement (see "Other Expenditures") with respect to this feature:

"In the case of highway and railway bridges over these waterways, the additional expenditures necessary to provide the type of structure and the horizontal and vertical clearances imposed in the interest of navigation have been considerable, and there is a continuing element of cost in lifting traffic to the elevation of many fixed bridges."

The charges referred to by Mr. Wonson are directly chargeable in a comparative analysis to the capital cost of the waterway. In most instances, were it not for the navigation requirement, all the bridges constructed over the part of the rivers considered in the Symposium could be reduced a distance of approximately 50 ft and still provide more than sufficient elevation to clear the highest high water. This reduction in height might result in a saving of approximately 50% of the capital cost of construction. Furthermore, the reduction, if allowed, would result in a program of new bridge construction over these rivers, and to consider, economically, the advantages potentially released thereby, would be incalculable.

Bridges have always been at a disadvantage when navigation requirements were the controlling economic factors. No attempt is ever made by the U. S. War Department to determine, economically, the relative value of the proposed bridge with respect to the vessels requiring increased clearance. These decisions are always in favor of water-borne traffic, and the War Department has always upheld any reasonable objection to reduction in clearance requirements. A bridge costing millions of dollars may never be constructed if some yachtsman using the waterway should require a clearance that would increase the cost of the structure to such an extent that the project would have to be abandoned.